DEVELOPMENT OF A MODEL TO EVALUATE CAPACITY OF URBAN MULTI-LANE ROADS UNDER HETEROGENEOUS TRAFFIC CONDITIONS

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Degree of Master of Philosophy

Department of Civil Engineering

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DECLARATION OF THE CANDIDATE & SUPERVISOR

I declare that this is my own work and this dissertation does not incorporate without acknowledgement any material previously submitted for a Degree or Diploma in any other University or institute of higher learning and to the best of my knowledge and belief it does not contain any material previously published or written by another person except where the acknowledgement is made in the text.

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DEDICATION

I dedicate this dissertation to Dr. H. R. Pasindu, my supervisor and mentor who encouraged me to complete this study successfully, and my parents who supported me throughout.

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10.05.2020

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ABSTRACT

Road capacity is defined as the maximum sustainable hourly flow rate at which vehicles can reasonably be expected to traverse a point or uniform section of a lane during a given time period under prevailing roadway and traffic conditions in the US Highway Capacity Manual. The knowledge of capacity of a given section of a road is an important input parameter for transport planning and traffic management studies. Presently, there aren't any up-to-date guidelines for road capacity estimation in Sri Lanka. The use of foreign guidelines is not recommended as each country has unique factors that influence capacity. Since urban multi-lane roads are typically the busiest roads, this research study focuses on developing a capacity estimation model for urban multi-lane roads in Sri Lanka.

Flow and speed data were collected using manual counting methods and Google Distance Matrix API (Application Program Interface) method respectively. The heterogeneous traffic flows were converted to Passenger Car Units (PCUs) using Chandra's method. Greenshields' traffic flow model was used to calibrate the empirical data. Capacity values were established from the developed flow-speed model. Using this method, the capacity values of all study locations were established. The average observed lane capacity was 1829 pcu/h/l.

Regression models were developed to estimate capacity of four-lane and six-lane roads. It was observed that the four-lane road capacity was influenced by the effective lane width, access point density, built environment and median type whereas the six-lane capacity was influenced by the effective lane width and access point density. The four-lane capacity model had an R-squared value of 0.81 and the six-lane capacity model had an R-squared value of 0.86. The two models were combined to create a single model that predicts both 4-lane and 6-lane roads. In addition to the capacity models, a regression model was developed to estimate the Free Flow Speed (FFS) of roads. The predictor variables of the FFS model are lateral clearance, built environment and median type. Verification of developed models were done by surveying 10 road sections. It was observed that all three models accurately predicted flow and speed from the statistical tests done (Mean Absolute Percentage Error <10%).

Important findings from the research study includes the development models to estimate four-lane and six-lane capacity values, and FFS. The typical base capacity for a 4-lane urban road was found to be 2044 pcu/h/l. The base capacity for a 6-lane sub-urban road section was estimated to be 2108 pcu/h/l. Even though the capacity values are comparable with capacity values in guidelines such as the HCM (1900-2200 pcu/h/l) since the speeds at capacity are in the range of 20km/h the traffic streams are susceptible to breakdown. The typical FFS of a rural road section with 2m lateral clearance and a center median was 50km/h. Sub-urban and urban road sections with similar conditions have 36km/h and 35km/h FFS speeds respectively. The findings of this research can be used for transport planning and traffic engineering studies in Sri Lanka as well as for further research in the area of capacity estimation.

Keywords: Capacity, urban roads, multi-lane roads, heterogeneous traffic, regression model, Free Flow Speed

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

Abbreviation	Description
PCU	Passenger Car Unit
LOS	Level of Service
HCM	Highway Capacity Manual
RDA	Road Development Authority
BPR	Bureau of Public Roads
FFS	Free Flow Speed
BFFS	Base Free Flow Speed
GDM	Google Distance Matrix
API	Application Program Interface
IHCM	Indonesian Highway Capacity Manual
TIRTL	The Infra-Red Traffic Logger
MAE	Mean Absolute Error
MAPE	Mean Absolute Percentage Error
PLM	Product Limit Method
RV	Recreational Vehicle
MC	Motorcycle
2W	Two-wheeler (Motorcycle)
TW/3W	Three-wheeler
LMV	Light Motor Vehicle
LV	Light Vehicle
HMV	Heavy Motor Vehicle
HV	Heavy Vehicle
SCV	Small Commercial Vehicle
LCV	Light Commercial Vehicle
MCV	Medium Commercial Vehicle
HCV	Heavy Commercial Vehicle
MAV	Multi Axle Vehicle
OSV	Oversized Vehicle

LIST OF APPENDICES

1 INTRODUCTION

1.1 Background

Sri Lanka has a rich road network of 12,220 km of A & B class highways, 170 km of Expressways (E class roads) and approximately 31,144 km in the entire road network [1]. A & B class roads along with E class roads are built according to higher design standards and are regularly maintained by the Road Development Authority (RDA). Of these roads, multi-lane roads cater to a major portion of the urban road network in the country. Therefore, the ability to quantify the traffic carrying capacities of these roads is essential in the development and maintenance of the road network. The Highway Capacity Manual (HCM), a guideline developed in the United States of America, defines capacity of a highway facility as the maximum hourly flow rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or a roadway during a given time-period under prevailing roadway and traffic conditions [2]. As implied by the definition, road capacity is a function of the roadway characteristics and the traffic stream.

Even though the definition of capacity is universal, since it is dependent upon roadway and traffic characteristics of a particular road section, the magnitude of the capacity varies with the locality. Capacity is a vital input parameter in most traffic and transportation related studies. Such as during the design and improvement of roads and its facilities where for example, the number of lanes, access control, median separation needs to be assessed. Capacity is also an important parameter in determining the Level of Service (LOS) of existing road sections. Further it is an input to BPR (Bureau of Public Roads) curves and similar demand functions used in planning studies.

At present the guideline used for capacity estimation in Sri Lanka is the Geometric Design Standards of Roads used by the Road Development Authority of Sri Lanka. This is a design guideline developed in the year 1998 based on the HCM of the year 1985. The issue in basing the local guideline on the HCM 1985 is that the HCM is developed in the United States of America where the conditions of the traffic flow and road geometry is vastly different from what is observed in Sri Lanka. Further the HCM itself has been subjected numerous revisions over the years with the latest edition being HCM published in 2010. At the start of this study, independent research done on the area of highway capacity locally was scarce. Jayaratne et al., (2016) estimated capacity of two-lane roads in Sri Lanka to be 2448 PCU/hr [3] where Passenger Car Unit (PCU) is a metric used to express the impact a given vehicle type has on traffic variables, in comparison to that of a passenger car.

At the international level varying amounts of research has been found to be done in the area of capacity evaluation. Countries such as Germany, China, Indonesia, Malaysia, Thailand, Denmark, Sweden, Australia have developed indigenous Highway Capacity Manuals whereas neighboring India is currently in the process of developing the Indo-HCM guideline. For example, Denmark derived modification factors to the US HCM methodology with steeper capacity reductions at non ideal conditions [4]. On the other hand, the Swedish capacity estimation method yielded higher capacity values than those given in the US HCM [5]. A study done by Velmurugan S. et al. (2014) used microscopic simulation to evaluate capacity for four-lane inter-urban highways in India which is a research done focusing the development of the Indo-HCM [6]. This research has significance to the current state of knowledge in the area of highway capacity evaluation, in evaluating how road capacities vary under heterogeneous traffic conditions and roadway characteristics which are prevailing in developing countries such as Sri Lanka.

Given the current status of research at the international level, it is clear that the level of research studies done locally in this area up to now is not adequate. The ability to accurately estimate capacity is fundamental in developing criteria to estimate the quality of service provided in urban road infrastructure as well as other roads. Further, avenues such as the impact various traffic and roadway characteristics have on capacity will be explored through this study.

1.2 Definitions

Capacity – Capacity is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway and traffic conditions. [7]

Multi-lane highway – A multilane highway is a road with two or more lanes in each direction of travel. There may or may not be a median strip or island between lanes of traffic moving in opposite direction. Different types of multi-lane highways are shown in Figure 1.1; (a) – Median separated multilane highway, (b) – A divided (by painted centerline) multilane highway, (c) – A two way right/left turn lane (TWRTL/TWLTL) where a common lane provided for cross-road movements, (d) – Another example of a divided multilane highway with painted centerline. A highway is not to be confused with an expressway which is an access-controlled road.

Figure 1.1: Different types of multi-lane roads [2]

Heterogeneous Traffic – Traffic streams that consist of vehicles with a wide range of static and dynamic characteristics with no spatial segregation are called Heterogeneous traffic streams. This is as opposed to homogeneous traffic streams where the majority of vehicles in the traffic stream are similar. Arasan and Krishnamurthy (2008) recommend that heterogeneous traffic mixes exist when the percentage of the dominant vehicle mode is less than 80% of the traffic mix [8], while Fazio, Hoque, and Tiwari (1999) suggest the value to be slightly higher at 85% [9]. The difference between a heterogeneous traffic stream (Figure 1.2-b) in Sri Lanka and homogeneous traffic stream (Figure 1.2-a) observed in the United States of America is shown in Figure 1.2.

Figure 1.2: Homogeneous traffic stream (a), and Heterogeneous traffic stream (b)

1.3 Scope of Study

Given that the existing knowledge on roadway capacity in Sri Lanka is minimal and the guidelines followed are quite outdated and not suited for local conditions, the requirement for a new guideline to estimate capacity is essential.

This study focuses on developing a model to estimate capacity of multi-lane highways in an urban setting. Urban multi-lane highways carry high traffic volumes and are an integral part of the road network. Through this research the traffic flow characteristics of multi-lane roads and how they relate with roadway characteristics are studied with the objective of developing a model to estimate roadway capacity.

1.4 Objectives

The objectives of the research are as mentioned below,

- Evaluation of the applicability of HCM 2010 methodology to analyse urban multi-lane road capacity in Sri Lanka (heterogeneous traffic conditions).

- To investigate the impact various traffic and roadway characteristics have on capacity of urban multi-lane roads in Sri Lanka.

- To develop a model to estimate the capacity values of urban multi-lane roads under heterogeneous traffic conditions considering roadway characteristics.

1.5 Outcomes

In this dissertation the following outcomes were generated,

- It was identified that the HCM 2010 multi-lane guideline was not applicable to estimate multi-lane capacity in Sri Lanka

- The roadway characteristics that influence multi-lane capacity were identified to be the Lane width, Median type, Access Point density and Built Environment.
- Separate and combined models to estimate 4-lane, 6-lane capacity were developed.
- A model to estimate FFS was developed and attached in APPENDIX D.

1.6 Arrangement of Dissertation

The structure of the dissertation is as follows,

Chapter one describes the background of the study including the suitability of existing guidelines for Sri Lankan road conditions, the scope of the study, objectives and outcomes.

Chapter two is the literature review of the study. The fundamentals of capacity and how the definition of capacity was assessed through time in different HCM guidelines are discussed in this chapter. Further, the US HCM 2010 multi-lane capacity assessment methodology, the Indonesian HCM multi-lane capacity assessment methodology and the RDA methodology are described. In addition to that the different capacity estimation models developed are discussed to assess which method is most suited to adopt for this study. Further, the literature on the impact of different traffic and roadway characteristics have on capacity are reviewed. Finally, the PCU derivation methods and data collection methods available are discussed in this chapter.

Chapter three explains three preliminary studies done in the overall study. First is the study done to evaluate the applicability of HCM 2010 multi-lane methodology to Sri Lankan conditions. Next is the pilot study done to test the capacity estimation methodology for the research. Finally, a comparative study is done to assess the best data collection technique for this research study.

Chapter four describes the data collection and capacity estimation components of the overall study. In this chapter a summary of the survey data collected is shown. Detailed location data are given in the appendix linked. Further, the finalized capacity estimation methodology is explained with an example. And finally, the capacity values obtained for all survey locations are presented.

Chapter five discusses the data analysis component of the study. This chapter has four subsections. Subsection 4 explains the visual trends observed in terms of the variation of capacity with different roadway characteristics. Subsection 5.2 describes the capacity model development procedure. Two linear regression models are developed to estimate four-lane and six-lane capacity based on roadway characteristics. These models are combined to form a single model capable of predicting both four-lane and six-lane capacities. In subsection 5.3 the traffic stream speed data related to capacity are discussed. Subsection 5.4 describes the verification process of the developed models. Subsection 5.5 compares the developed capacity model performance with existing models in literature and Subsection 5.6 describes the limitations of the study.

Chapter six concludes the research thesis with a summary of the research findings and recommendations for further studies in the research area.

2 LITERATURE REVIEW

2.1 Fundamentals of Capacity

2.1.1 Capacity Definitions through time

The definition of capacity has been evolving with time since it was initially introduced in the early 20th century. The HCM 2010 which is the latest edition of the Highway Capacity Manuals published by the Transport Research Board of the USA defines capacity as follows,

"The capacity of a system element is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway and traffic conditions."

In search of getting a clear understanding about capacity Roess and Prassas asks in the following questions in their 2014 book "The Highway Capacity Manual: Conceptual and Research History" [10]:

1. In what units is capacity to be measured?
2. Over what period of time is capacity to be measured?
3. How should the characteristics of the highway be defined, and what characteristics of the highway will affect the value of capacity?
4. What operating characteristics define the occurrence of capacity?
5. How should the characteristics of the traffic using the highway be defined, and what characteristics of traffic will affect the value of capacity?

This provides a good starting point to analyze and understand the concepts of capacity. How the understanding of capacity and its definition transformed over time is documented below.

2.1.1.1 From years 1920 to 1950

During the formative years of the concept of capacity in **1930** by A. N. Johnson, the following was noted,

"We can visualize a road carrying but a few vehicles and agree that there is no congestion. But as the number of vehicles increases, there will be a point reached at which some vehicles will be delayed because they are immediately unable to pass slower-moving vehicles. Such a point indicates the beginning of congestion or what may be called 'working capacity' or 'free-moving capacity' of the highway." [11]

The usage of the term "capacity" in this definition means the start of a traffic jam. The terms "working capacity" or "free moving capacity" are used to indicate the point at which the maneuverability of an individual vehicles get effected by the other vehicles in the traffic stream. At this point in time capacity did not mean the maximum flowrate of vehicles even though the units for capacity were vehicles per hour (volume).

The now popular flow-speed-density diagrams were first developed by Bruce D. Greenshields during this era. He was the first person in 1934 to represent capacity in terms of a maximum value of a calibrated speed-flow curve (shown in Figure 2.1) [12]. The data set (small percentage of trucks included) had a capacity value of approximately 2180veh/h/l.

2.1.1.2 The 1950 Highway Capacity Manual

Highway capacity was formally defined for the first time in the Highway Capacity Manual of 1950 [13]. The HCM 1950 defined three capacity values as shown below,

> ***Basic Capacity**: Basic capacity was defined as the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained.*

Possible Capacity*: Possible capacity was defined as the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under the prevailing roadway and traffic conditions.*

Practical Capacity*: Practical capacity was defined as the maximum number of vehicles that can pass a given point on a roadway or lane during one hour without the traffic density being so great as to cause unreasonable delay, hazard, or restriction to the drivers' freedom to maneuver under prevailing roadway and traffic conditions.*

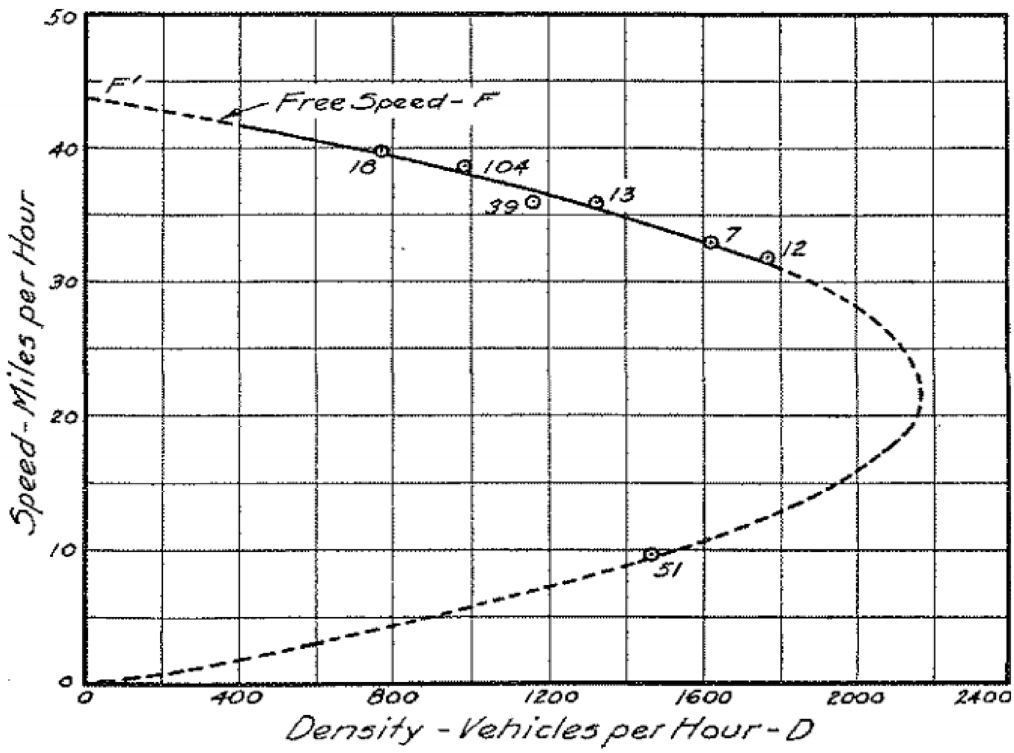


Figure 2.1: Greenshields' speed-flow curve in 1934 [12]

Basic capacity was defined as the capacity when the road and traffic conditions were "nearly ideal". The manual does not define what these ideal conditions are, but it is understood that the ideal conditions are similar to what are considered ideal conditions at present. Such as having 3.6m lane widths, 1.8m lateral clearances, level terrain, no slow-moving vehicles in the traffic stream etc.

Roadway conditions included features such as the horizontal and vertical alignment, which are generally represented in terms of the design speed of the highway. Traffic conditions expressed the presence of slow-moving vehicles primarily at this point in time. In later editions the differences between weekday and weekend drivers were also considered.

While possible capacity included leeway for reduction in capacity due to prevailing roadway and traffic conditions in the examined one-hour period, practical capacity touched on the subject of Level of Service (LOS) in a manner in its definition for the first time.

Table 2.1: Capacity values given in in 1950 HCM [13]

Type of Facility	Basic Capacity	Possible Capacity	Practical Capacity
Rural multi-lane highway	2000 pcu/h/l	See note	1000 pcu/h/l
Urban multi-lane highway	2000 pcu/h/l	See note	1500 pcu/h/l

Note: Possible capacity is the basic capacity excluding the negative impact of prevailing conditions

2.1.1.3 The 1965 Highway Capacity Manual

The three capacity types definition was replaced by a single one in the 1965 HCM as below,

"Capacity is the maximum **number of vehicles** which has a **reasonable expectation** of passing over a given section of a lane or a roadway in one direction (or in both directions for a two-lane or three-lane highway) during a given time period under prevailing roadway and traffic conditions." [14]

The definition given in the 1965 HCM is closely related to “Possible Capacity” in the 1950 HCM. A noteworthy addition to the capacity definition was the addition of the term “reasonable expectation”, which meant that the it was recognized at this point in time that capacity is not a static, but a value that could vary depending upon factors that are hard to quantify such as the driver behavior [10].

The 1965 HCM introduced four constrains that affect the flow of vehicles across a point. The capacity will be controlled by one of the four factors,

> “1. *The demand based upon vehicles whose drivers/passengers desired to use the roadway during the time of observation,*
> *2. The capacity of the location of observation,*
> *3. The capacity at a point upstream of the of the location of observation, or*
> *4. The capacity at a point downstream of the location of observation.”*

This was an interesting development in the evolution of highway capacity since with this knowledge the capacity of interrupted and uninterrupted flows can be separately understood.

Further the term Level of Service (LOS) was introduced in the 1965 HCM with six LOS categories from ‘A’ to ‘F’. ‘A’ being the best LOS and ‘F’ being the worst. The 1965 HCM can be considered as one of the pioneering documents in introducing modern day capacity concepts to the world.

2.1.1.4 The 1985 Highway Capacity Manual

With an increase in research studies in the area of capacity, the concept of capacity defined in the 1985 manual was close to what is in use at present,

> *“In general, the capacity of a facility is the maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point or*

uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions." [15]

Considering the definition given in the 1985 HCM it was noted that for the first-time capacity is coined in terms of a flow rate. This is one of the major updates observed in the 1985 HCM over the 1965 HCM. The manual defines that a 15-minute time interval is the period used to define capacity flow rate.

Another addition observed in the 1985 HCM was that ideal or base capacity was separated in terms of design speed. Different design speeds (which vary based on the alignment parameters of the section) had unique base capacity values.

The list of base conditions was changed slightly in the 1985 HCM, but not to a significant degree. For multilane highways, the following were included:

- 12ft (3.6m) lane width,
- 6ft (1.8m) lateral clearance,
- No heavy vehicles (trucks, buses, RVs) in the traffic stream.

The base capacity values in the 1985 HCM are shown in Table 2.2. A freeway

Table 2.2: Base capacity values in 1985 HCM [15]

Facility Type	Ideal or Base Capacity
Freeway[1], Multilane (97, 113 km/h)	2000 pc/h/l
Freeway, Multilane (80 km/h)	1900 pc/h/l
Two-Lane, Two-Way	2800 pc/h/l
Signalized Intersections	1800 pc/hg/l

[1] Freeways are defined as separated highways with full control of access and two or more lanes in each direction dedicated to the exclusive use of traffic

2.1.1.5 The Interim Updates: 1994 and 1997

With a high number of research done in the United States of America following the release of the 1965 HCM it was observed that the capacity of both multi-lane highways and freeways exceeded 2000 pcu/h/l value given in the 1965 HCM [10]. This phenomena was observed in independent research done during the time as well [16, 17, 18, 19].

Given the volume of research, two interim updates to the HCM was introduced in 1994 and 1997. Of these two, the 1994 update revised the multi-lane capacity estimation methodology by introducing a new parameter called Free Flow Speed (FFS) [15]. Theoretically, FFS is the speed of the traffic stream at ‘0’ density (veh/km) [10]. According to the 1994 update to the 1985 HCM, the lane width, median type, lateral clearance, and access point density affected the FFS of a multi-lane highway [15]. Table 2.3 is an extract from the 1994/1997 HCM depicting the revised capacity values.

Table 2.3: Base capacity values in 1994 and 1997 HCM [15]

Type of facility	Base Capacity in:	
	1994 Update	1997 Update
4-lane Freeways:		
FFS = 113, 121 km/h	2200 pc/h/l	2400 pc/h/l
FFS = 105 km/h	2200 pc/h/l	2350 pc/h/l
FFS = 97 km/h	2200 pc/h/l	2300 pc/h/l
FFS = 89 km/h	2200 pc/h/l	2250 pc/h/l
6- or 8-lane Freeways:		
FFS = 113, 121 km/h	2300 pc/h/l	2400 pc/h/l
FFS = 105 km/h	2300 pc/h/l	2350 pc/h/l
FFS = 97 km/h	2300 pc/h/l	2300 pc/h/l
FFS = 89 km/h	2300 pc/h/l	2250 pc/h/l
Multilane Freeways:		
FFS = 97 km/h	2200 pc/h/l	2200 pc/h/l

Table 2.3: Base capacity values in 1994 and 1997 HCM (Continued)

FFS = 89 km/h	2100 pc/h/l	2100 pc/h/l
FFS = 80 km/h	2000 pc/h/l	2000 pc/h/l
FFS = 72 km/h	1900 pc/h/l	1900 pc/h/l

2.1.1.6 The 2000 Highway Capacity Manual

The fundamental definition of capacity remained unchanged in this edition. The capacity values for multi-lane highways were not revised either. However, the manual included a better definition of the concept of "reasonable expectancy" when defining capacity:

"Reasonable expectancy is the basis for defining capacity. That is, the stated capacity for a given facility is a flow rate that can be achieved repeatedly for peak periods of sufficient demand. Stated capacity can be achieved on facilities with similar characteristics throughout North America." [20]

2.1.1.7 The 2010 Highway Capacity Manual

The definition of capacity in the 5th edition of the HCM is the same as in previous editions since 1985:

"The capacity of a system element is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway and traffic conditions." [2]

The base capacity values of multi-lane highways too have not changed in the 2010 HCM meaning that they have not been altered since 1997.

2.1.2 Development of HCM Multi-lane Methodology through time

No formal definition for a multi-lane highway is given in the HCM 2010. But typically, multi-lane roads are roads with at least 2 lanes of traffic in each direction of travel. The HCM however, provides guidance on whether a multilane road is operating under uninterrupted flow:

"In general, uninterrupted flow may exist on a multilane highway if there are 2 miles (3.2km) or more between traffic signals."

Multilane highways, however, may have un-signalized intersections and driveways. Portions of multilane highways that are more than 2 miles away from the nearest signalized intersection are said to operate under uninterrupted flow [7]. Table 2.4 presents a summary of the adjustment factors and capacity values given the HCM over time. It is seen that throughout the evolution of the HCM the base capacity is hovering around the 2000 pcu/h/l value.

Table 2.4: Summary of HCM capacity values and adjustment factors

HCM Edition	Type of Facility	Type	Capacity (pc/h/ln)	Adjustment factors
1950	Multilane highway	Basic Capacity	2000	• Lane width and lateral clearance • Heavy vehicle adjustment
		Practical Capacity – Urban	1500	
		Practical Capacity - Rural	1000	
1965	Multilane highway	Base capacity	2000	• Lane width and lateral clearance • Heavy vehicle adjustment
1985	Multilane highway	Design speed > 80km/h	2000	• Lane width and lateral clearance • Heavy vehicle adjustment • Development environment • Driver population
		Design speed = 80km/h	1900	
1994	Multilane highway	FFS – 97 km/h	2200	• Median type • Lane width • Lateral clearance • Access-point density
		FFS – 89 km/h	2100	
		FFS – 80 km/h	2000	
		FFS – 72 km/h	1900	
1997, 2000, 2010	Multilane highway	FFS – 97 km/h	2200	• Median type • Lane width • Lateral clearance • Access-point density • Heavy vehicle adjustment • Driver population
		FFS – 89 km/h	2100	
		FFS – 80 km/h	2000	
		FFS – 72 km/h	1900	

2.1.3 “Capacity Drop” Phenomena

Capacity drop is a phenomenon observed when comparing the flow capacity values before and after a breakdown in a traffic stream [10]. That is, the capacity value upstream of a breakdown and the capacity value downstream of breakdown (queue discharge). Two researchers, Ellis [21] and Eddie [22] observed that the capacity value on the unstable flow side (upstream of breakdown) is considerably higher than the capacity value on the stable side of the flow (downstream of breakdown). A study carried out by Drake et. al [23] concluded that such a phenomenon is observed and that stable flow values could only be measured at or immediately after a breakdown and unstable flow values can only be measured upstream of a breakdown. Banks et. al [24] proposed a multi-regime traffic flow model based on the “capacity drop” observed after a breakdown in flow. They observed that the upstream flow is slightly higher than the downstream flow.

A research carried out by Agymang-Duah and Hall [25] showed that from a sample of 52 sections approximately 10% of the time the stable capacity flow values were higher than those on the unstable side. Hence it cannot always be considered a “drop” and there is a randomness to its variation. Further, in a study done by Li & Laurence [26] in 2015, they observed that a capacity drop was observed on only 9 out of 112 days surveyed.

2.1.4 Defining capacity based on Empirical data

Capacity is a value derived based on empirical data. Hence it has a random variation to it. Therefore, even though given as a static value the actual capacity will change due to factors that aren’t easily quantifiable. Roess and Prassas [10] argue that since capacity is expressed in terms of “reasonable expectation” the given value should be in the low end of the capacity distribution, i.e. 15^{th} percentile. The mean or the 50^{th} percentile value or the 85^{th} percentile value of the distribution can be selected but this would not satisfy the “reasonable expectancy” given in the definition of capacity by the Highway Capacity Manual.

2.2 HCM 2010 Multi-lane Methodology

The methodology given in the HCM 2010 to estimate multi-lane capacity is reviewed in this section.

2.2.1 Limitations of HCM 2010 Methodology

The HCM methodology does not consider the following factors when estimating capacity [7],

- Impacts due to inclement weather, accidents, crossings etc.
- Significant presence of on-street parking
- Effect of the change in number of lanes
- Effects due to different types of medians
- FFS below 72 km/h (45mi/h) or higher than 96km/h (60mi/h)
- Presence of bus stops with high passenger turn over
- Significant pedestrian activity

Factors such as the presence of on street parking, bus stops, significant pedestrian activity are more prominent in highways in Sri Lanka. Hence these factors are expected to have an impact on the flow of traffic. Further, the HCM 2010 caters to traffic flows having FFS between 72 km/h to 97 km/h. Such speeds are not expected on Sri Lankan roads and studies need to be carried out to get an understanding about FFS values of local multi-lane roads.

2.2.2 Base Conditions for Multi-lane Highways in HCM 2010 [7]

The following parameters should be satisfied for a section to be considered ideal or 'base' according to the HCM 2010,

- Lane width – 3.6m
- Lateral clearance – 1.8m

- Access point density – zero
- Median – available
- Terrain type – level
- Zero heavy vehicle presence in traffic stream
- Driver population comprising of users familiar with the highway

The base conditions may vary from country to country depending on the local conditions.

2.2.3 HCM 2010 Capacity Estimation Framework

HCM 2010 defines capacity of a multi-lane highway with respect to the FFS of passenger cars on the road section under base conditions. Hence the FFS acts as an intermediary between the factors that affect capacity and capacity. Figure 2.2 depicts the speed-flow curve given by the HCM 2010. Separate curves are given for each base FFS from 45mi/h (72 km/h) up to 60mi/h (97km/h) with corresponding capacity values. The methodology defines capacity for a range of FFS citing that it is subjected to vary widely. It is recommended to estimate FFS to the nearest 5 mi/h (8km/h). Further, it is said that speeds of passenger cars will remain constant up to 1400pc/h/ln (cars will move at FFS) and decline only when the flowrate increases further.

Table 2.6 presents the respective FFSs, their speed range, their corresponding capacity values and the speeds at capacity. The speeds at capacity are not less than 10km/h of the FFS which is an interesting observation and provides an insight into the type of traffic streams the manual caters to.

The curves indicated in Figure 2.2 is described by the equations given in Table 2.5.

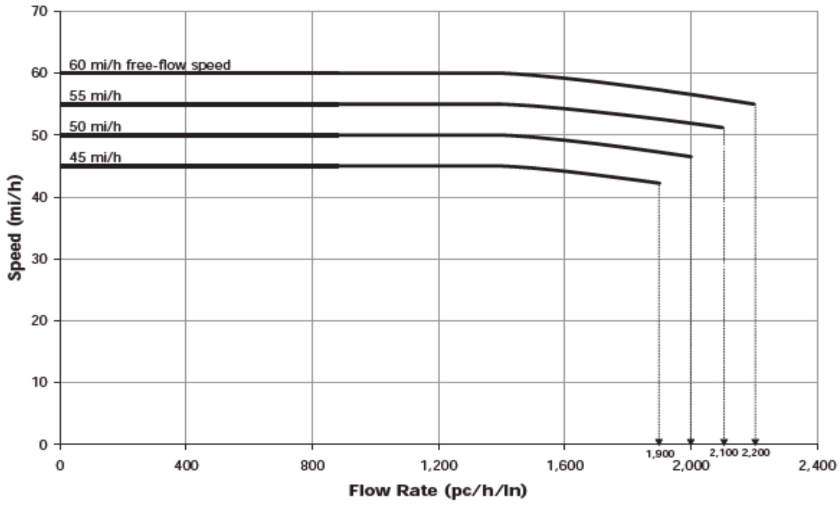


Figure 2.2: HCM 2010 speed-flow curve for multi-lane highways under base conditions [7]

Table 2.5: Equations describing speed-flow curves in Figure 2.2 [7]

FFS (mi/h)	For $v_p \leq 1{,}400$ pc/h/ln, S (mi/h)	For $v_p > 1{,}400$ pc/h/ln, S (mi/h)
60	60	$60-\left[5.00\times\left(\frac{v_p-1400}{800}\right)^{1.31}\right]$
55	55	$55-\left[3.78\times\left(\frac{v_p-1400}{700}\right)^{1.31}\right]$
50	50	$50-\left[3.49\times\left(\frac{v_p-1400}{600}\right)^{1.31}\right]$
45	45	$45-\left[2.78\times\left(\frac{v_p-1400}{500}\right)^{1.31}\right]$

Table 2.6: FFS values and respective capacity values

FFS (mi/h)	FFS (km/h)		Capacity (PC/h/ln)	Speed at capacity (km/h)
		Range		
45	72	68≤FFS<76	1900	68
50	80	76≤FFS<84	2000	75
55	89	84≤FFS<93	2100	82
60	97	93≤FFS<101	2200	89

2.2.4 Computation of Free Flow Speed (FFS)

The HCM 2010 methodology provides two methods to compute FFS.

1. By Field Measurement

By this method the mean speeds of cars will be measured at low to moderate flows (up to 1400pc/h/ln). No adjustments are required to be applied to the measured speed. The speed survey is to be carried out at a location representative of the road segment. The study should either measure the speeds of all cars or use a systematic sample (e.g., every 10^{th} car in each lane) for the measurement. A sample of at least 100 car speeds should be obtained. Any method of speed determination accepted for other types are of traffic engineering applications, are said to be accepted. Further, the manual considers this method to be the more accurate method of calculating FFS.

2. By Estimation using equation

The second method to compute FFS is by estimation based on the physical characteristics of the segment under consideration. The FFS estimated using Equation (2.1) [7].

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A \qquad (2.1)$$

where,

BFFS = Base FFS for multilane highway segment (mi/h);

FFS = FFS of basic multi-lane segment (mi/h);

f_{LW} = Lane width adjustment (mi/h);

f_{LC} = Adjustment for total lateral clearance (mi/h);

f_M = Adjustment for median type (mi/h);

f_A = Adjustment for access point density (mi/h)

The Base Free Flow Speed (BFFS) is the most significant value in equation (2.1). The methodology presents two ways to determine the BFFS of a segment. One way is to use the design speed of the highway as the BFFS. The other is to use the posted speed of the segment with an adjustment factor as shown below,

For posted speed 50mi/h (80 km/h) and higher BFFS = posted speed + 5mi/h (8 km/h)
For posted speed below 50mi/h (80 km/h) BFFS = posted speed +7mi/h (11 km/h)

2.2.5 Adjustment Factors

Lane width adjustment

The base condition for the lane width is 12ft (3.6m) or greater. If the lane with is less than the base condition it negatively affects the FFS. Here the manual assumes that vehicles will travel in their designated lanes without using additional lane width if available for travel. This is true for homogeneous traffic streams. However this is not the case for heterogeneous traffic streams [8]. Table 2.7 shows the adjustment factors given in the HCM 2010 manual. Equivalent values for 12ft, 11ft, and 10ft in meters are 3.6m, 3.35m and 3m. The manual does not cater to roads with lane width less than 3m.

Table 2.7: Lane width reduction factors for equation (2.1) [7]

Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Adjustment for Lateral Clearance

The sum of the lateral clearance on the left and right sides of the road is considered here. Both periodic and continuous obstructions are taken into consideration under this adjustment factor. All structures other than raised curbs (see Figure 2.3) are considered as obstructions. The base condition here is to have a lateral clearance of at least 6ft

(1.8m) on either side of the road (Total 12ft). Right side obstructions (the median) are subjected to some judgement as most barrier types do not affect the driver behavior. Table 2.8 shows the adjustment factors given in the HCM 2010 for lateral clearances for both four lane and six lane highways. (12ft = 3.6m, 10ft = 3m, 8ft = 2.4m, 6ft = 1.8m, 4ft= 1.2m, 2ft= 0.6m)

Figure 2.3: Image of curb (source: gettyimages)

Table 2.8: Adjustments to FFS for lateral clearances given in HCM 2010 [7]

Four-Lane Highways		Six-Lane Highways	
TLC (ft)	Reduction in FFS (mi/h)	TLC (ft)	Reduction in FFS (mi/h)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

Note: Interpolation to the nearest 0.1 is recommended.

<u>Adjustment for median type</u>

Table 2.9 shows the adjustment factors given in the HCM 2010 to the FFS based on the median type of a highway section. A reduction factor is proposed if no median is available to separate opposing traffic lanes. (TWLTL is a two-way left turn lane which is not found in Sri Lanka)

Table 2.9: Adjustment factor to FFS for Median type in HCM 2010 [7]

Median Type	Reduction in FFS, f_M (mi/h)
Undivided	1.6
TWLTL	0.0
Divided	0.0

Adjustment for Access point density

Access points are defined as driveways and un-signalized intersections present on the left side of the highway. If access points with little activity are present, they are not be included to the calculation. Table 2.10 shows the table given in HCM 2010.

Either through field measurement or by using equation (2.1) the FFS value is calculated. Using the obtained FFS value a speed-flow curve should be selected from the chart shown in Figure 2.2. This will provide the user with one of four capacity values based on the FFS. This value is the estimated capacity for the assessed highway section.

Table 2.10: Adjustment to FFS for access point density in HCM 2010

Access-Point Density (access points/mi)	Reduction in FFS, f_A (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
≥40	10.0

Note: Interpolation to the nearest 0.1 is recommended.

The HCM 2010 further provides the user with the option to estimate the LOS of a given highway section by examining the flow of vehicles on the highway section. Adjustment factors for the percentage of trucks, buses, RVs, longitudinal grade, and type of driver population are provided [7].

2.3 Indonesian HCM 1997 (IHCM) Urban Multi-lane Methodology

The Indonesian Highway Capacity Manual is a guideline developed for Indonesian traffic conditions primarily based on the US-HCM 1994. The guideline has separate capacity estimation methodologies for two lane and multilane highways as well for both urban (and semi-urban) and rural road sections. A review of the urban multi-lane capacity estimation methodology is given below.

The IHCM 1997 defines capacity as 'the maximum stable traffic flow that can be maintained under available geometric, environmental and traffic conditions' [27].

2.3.1 Limitations and Base Conditions

The IHCM defines a set of conditions that needs to be met to use its methodology similar to the HCM.

<u>The base conditions for four-lane divided road (4/2 D)</u>

- Lane width of 3.5m (total width of traffic lanes 14m)
- Curb (no shoulder)
- The distance between curb and the nearest barrier on the sidewalk ≥ 2m
- Median
- Low side friction
- City size 1.0 to 3.0 million
- Flat alignment

<u>The base conditions for a four-lane undivided (4/2 UD)</u>

- Lane width of 3.5m (total width of traffic lanes 14m)
- Curb (no shoulder)
- The distance between curb and the nearest barrier on the sidewalk ≥ 2m
- Median
- Low side friction
- City size 1.0 to 3.0 million

- Flat alignment
- Directional split 50-50

2.3.2 IHCM Capacity model

The model to determine capacity given by the IHCM is as follows,

$$C = C_0 \ x \ FC_W \ x \ FC_{SP} \ x \ FC_{SF} \ x \ FC_{CS} \quad (2.2)$$

Where,

C = Capacity (pcu/h)

C_0 = Base Capacity (pcu/h)

FC_w = Lane width adjustment factor

FC_{SP} = Directional split adjustment factor (for undivided roads)

FC_{SF} = Side friction adjustment factor and shoulder / curb adjustment

FC_{CS} = City size adjustment factor

Base Capacity (C_0)

The base capacity details are shown in Table 2.11. The IHCM 1997 differentiates between divided and undivided roads and provides different capacity values for each type of road.

Table 2.11: Base capacities given in IHCM 1997 [27]

Type of Road	Base Capacity
Four-lane divided/ One-way road	1650 pcu/h/l
Four-lane undivided	1500 pcu/h/l
Two-lane undivided	2900 pcu/h/dir

Adjustment factor for lane width (FC_W)

The lane width which is the width of road available for the use of the motorists is one of the factors stated to have an impact on capacity by the IHCM. The adjustment factors for capacity based on the lane width is shown in Table 2.12. While both the HCM and IHCM has 3m as the minimum lane width the IHCM has positive adjustment factors from its base lane width of 3.5m up to 4m. This is an acknowledgement to the heterogeneous nature of traffic observed in Indonesia.

Table 2.12: Lane width adjustment factors for urban roads [27]

Type of Road	Lane width (m)	FC_W
Four-lane divided or One-way	3.00	0.92
	3.25	0.96
	3.50	1.00
	3.75	1.04
	4.00	1.08
Four-lane undivided	3.00	0.91
	3.25	0.95
	3.50	1.00
	3.75	1.02
	4.00	1.09
Two-lane undivided	5	0.56
	6	0.87
	7	1.00
	8	1.14
	9	1.25
	10	1.29
	11	1.34

Capacity adjustment factor for directional split (FC_{SP})

Table 2.13 shows the adjustment factors for capacity due to the directional split of vehicles on undivided roads. For divided roads $FC_{SP} = 1$.

Table 2.13: Directional split adjustment factors for capacity [27]

Directional Split		50-50	55-45	60-40	65-35	70-30
FC_{SP}	Two-lane	1	0.97	0.94	0.91	0.88
	Four-lane	1	0.99	0.97	0.96	0.94

Side friction adjustment factor (FC_{SF})

The side friction factor is a complex factor introduced by the IHCM to incorporate factors that are commonly observed on the side of the road in Indonesia. This includes pedestrian movement, vehicle interactions with the traffic flow, slow moving vehicles in addition to the width of the shoulder or curb.

a) Road width shoulders

The adjustment factor for roads with shoulders are shown in Table 2.14 below. The side friction class is to be taken from Table 2.15.

Table 2.14: Capacity adjustment factor for the effect of side friction and shoulder width [27]

Type of Road	Side friction class (SFC)	Adjustment factor for side friction class and shoulder (FC_{SF})			
		Effective shoulder width (W_S)			
		<0.5	1.0	1.5	≥ 2.0
Divided four-lane road	VL	0.96	0.98	1.01	1.03
	L	0.94	0.97	1.00	1.02
	M	0.92	0.95	0.98	1.00
	H	0.88	0.92	0.95	0.98
	VH	0.84	0.88	0.92	0.96
Undivided four-lane road	VL	0.96	0.99	1.01	1.03
	L	0.94	0.97	1.00	1.02
	M	0.92	0.95	0.98	1.00
	H	0.87	0.91	0.94	0.98
	VH	0.80	0.86	0.90	0.95
Undivided two-lane road or One-way street	VL	0.94	0.96	0.99	1.01
	L	0.92	0.94	0.97	1.00
	M	0.89	0.92	0.95	0.98
	H	0.82	0.86	0.90	0.95
	VH	0.73	0.79	0.85	0.91

Side Friction Class (SFC)

The frequency of the following per hour for 200m on both sides along the segment is used to compute the SFC. The weight each factor holds is given within brackets. The weighted total of the four factors given below will determine the SFC from Table 2.14.

- Number of pedestrians walking along or crossing the road segment (0.5)
- Number of vehicles stopped and parked (1.0)
- Number of vehicles that enter and exit the road from the roadside (0.7)
- Slow moving vehicle flows. The total flow of (veh/h) on a bicycle, tricycle, wagons, tractors etc. (0.4)

Table 2.15: Side Friction Class table IHCM [27]

Side Friction Class	Code	The weighted number of incidents per 200m per hour	Conditions
Very Low	VL	<100	Residential areas. Side streets present.
Low	L	100-299	Residential areas. Some public transport etc.
Moderate	M	300-499	Industrial areas. Shops on side streets.
High	H	500-899	Commercial area. High street side activity.
Very High	VH	>900	Commercial area with market activities beside the road.

b) Road with Curb

The adjustment factor for roads with curbs are shown in Table 2.16 below. The side friction class is to be taken from Table 2.15.

Table 2.16: Capacity adjustment factor for the effect of side friction and curb width [27]

Type of Road	Side friction class	Adjustment factor for side friction class and shoulder			
		Curb width			
		<0.5	1.0	1.5	≥ 2.0
Divided four-lane road	VL	0.96	0.97	0.99	1.01
	L	0.94	0.96	0.98	1.00
	M	0.91	0.93	0.95	0.98
	H	0.86	0.89	0.92	0.95
	VH	0.81	0.85	0.88	0.92
Undivided four-lane road	VL	0.95	0.97	0.99	1.01
	L	0.93	0.95	0.97	1.00
	M	0.90	0.92	0.95	0.97
	H	0.84	0.87	0.90	0.93
	VH	0.77	0.81	0.85	0.90
Undivided two-lane road or One-way street	VL	0.93	0.95	0.97	0.99
	L	0.90	0.92	0.95	0.97
	M	0.86	0.88	0.91	0.94
	H	0.78	0.81	0.84	0.88
	VH	0.68	0.72	0.77	0.82

Adjustment factor for City size (FC_{CS})

The IHCM has entered a factor called city size factor which takes into account the population of the city in which the road analyzed is present. According to the factor the capacity will be reduced with the reduction in the population. The factors are shown in Table 2.17.

Table 2.17: City size adjustment factor in IHCM [27]

City size (population in millions)	Adjustment factor
<0.1	0.86
0.1-0.5	0.90
0.5-1.0	0.94
1.0-3.0	1.00
>3.0	1.04

Capacity adjustment factor for six lane roads

To determine capacity of six lane roads the methodology provides an adustment factor in the form equation to the side friction factor of four lane roads. The equation is shown in equation (2.3) below.

$$FC_{6,SF} = 1 - 0.8(1 - FC_{4,SF}) \tag{2.3}$$

Based on the four attributes explained above the IHCM 1997 predicts the capacity of multi-lane (four-lane and six-lane roads). With a base capacity of 1650 pcu/h/l the maximum achievable lane capacity for a four-lane road according to equation (2.2) is 1909 pcu/h/l.

1997 IHCM PCU factors for Urban roads

The IHCM 1997 provides PCU factors for motorcycles and heavy vehicles to convert the actual flow into a homogeneous flow. This is used for the purpose of comparing existing traffic flows with capacity to get an understanding about the LOS of a road section. The factors are given in Table 2.18.

Table 2.18: IHCM 1997 PCU factors for Urban roads

Type of road	Traffic flow per lane (veh/h/l)	PCU	
		Heavy vehicles	Motorcycles
Two-lane one way and four-lane divided	0	1.3	0.4
	≥1050	1.2	0.25
Three-lane one way and six-lane divided	0	1.3	0.4
	≥1050	1.2	0.25

The IHCM 1997 provides a separate methodology to calculate FFS unlike the HCM methodology where FFS and Capacity are interrelated.

2.4 RDA Methodology (1998)

The Geometric Design Standard for Roads published by the Road Development Authority (RDA) Sri Lanka uses the HCM 1985 guideline for multilane capacity calculations. The capacity for an uninterrupted flow segment of a multilane road is given as 2000 pcu/h/l [28].

In conclusion it is seen that the base capacity values of different guidelines are around the value of 2000pcu/h/l. A summary of the details are given in Table 2.19.

Table 2.19: Summary of base capacity values given in guidelines

Guideline	Base Capacity (pc/h/ln)
HCM 2010	2200 (FFS = 100km/h)
IHCM	1650
AustRoads	2200
RDA Guideline	2000 (HCM 1985)

2.5 HCM 2010 Urban Street Segments methodology

The Urban Street Segments methodology of the 2010 HCM describes a methodology to evaluate the capacity and performance of urban streets. This methodology is applicable to two-lane and multi-lane urban roads. An urban street segment is defined as a length of road which is less than 2 miles (3.2km) in length and bounded by an intersection or ramp terminal. The analysis boundaries of an urban street segment are the upstream and downstream intersections of a road link and the Right of Way (ROW) of the road. Figure 2.4 depicts a typical urban road segment.

The performance measures of the LOS of an urban street segment are the speed of through vehicles and the volume-to-capacity ratio of the through movement in the downstream intersection. Hence, the capacity of an urban street segment is said to be dictated by the intersection capacity. The factors considered for performance evaluation in the methodology include the number of access points, the presence of a median separation, presence of a curb, number of lanes and turning bays (and length). The performance evaluation methodology provided incorporates the dynamics of all these components along with the intersection characteristics of the segment boundaries. Whilst the applicability of this methodology to Sri Lankan conditions

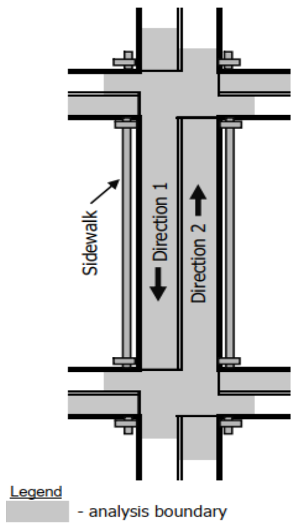


Figure 2.4: Analysis boundary of an Urban Street segment in HCM 2010 [7]

needs to be reviewed in depth, since the methodology does not shed any light on midblock traffic capacity evaluation it is not reviewed further.

2.6 Capacity Estimation models and methods

During the early years of capacity research, the focus was on the relationship between vehicle speeds and the spacing between vehicles [10]. Spacing was defined as the distance between the centers of two consecutive vehicles in a traffic stream. The argument was that with the increase in speed the spacing between vehicles will increase to accommodate the reaction time and the breaking distance in addition to the standstill distance between vehicles.

Once the spacing between vehicles in a traffic stream was established, the density and hence the flow rate of vehicles can be calculated using the fundamental traffic flow equation shown in (2.4),

$$Q = U\ x\ K \tag{2.4}$$

Where,

Q = flow rate of vehicles (veh/h)

U = Speed (km/h)

K = Density (veh/km)

2.6.1 Greenshields' model

Greenshields published the first major empirical model in the field of traffic engineering in 1935 [12]. The model which depicted the association between speed and density is shown in equation (2.5) below,

$$v = v_f - \left[\frac{v_f}{k_j}\right] k \tag{2.5}$$

Where,

v_f = Speed at '0' density (km/h)

k_j = jam or maximum density (veh/km)

v = Speed (km/h)

k = Density (veh/km)

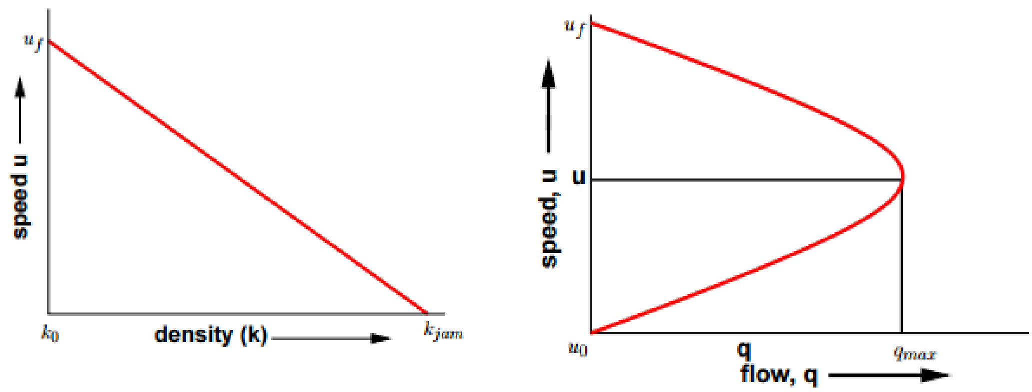


Figure 2.5: Speed-density and speed-flow model developed by Greenshields [12]

Even though this model was based on extensive data of two-lane highways it has been successfully used for multilane highways and freeways as well in later studies [10]. A point to note is that Greenshields derived this model while studying normal or in other words 'not ideal' conditions as is done at present. The 1935 paper deduces that the capacity of a single lane under uninterrupted flow section as 2,180 veh/h – which was rounded to 2,200 veh/h by Greenshields.

2.6.2 Greenberg's model

Greenberg in a paper published in 1959 hypothesized that traffic flow would conform to the general characteristics of fluid flow [29]. The developed model (equation (2.6)) is of logarithmic shape.

$$v = v_c ln\left(\frac{k_j}{k}\right) \tag{2.6}$$

Where,

v = Average speed (km/h)

v_c = Speed at capacity (km/h)

k = Density (veh/km/ln)

k_j = Jam density (veh/km/ln)

The principal drawback with the logarithmic curve developed by Greenberg was that as the model density approaches "0", speed moves asymptotically to infinity (see Figure 2.6). Many subsequent researchers have suggested to separately calibrate an FFS (maximum speed) and 'anchor' the curve to this value as a solution to the issue [10].

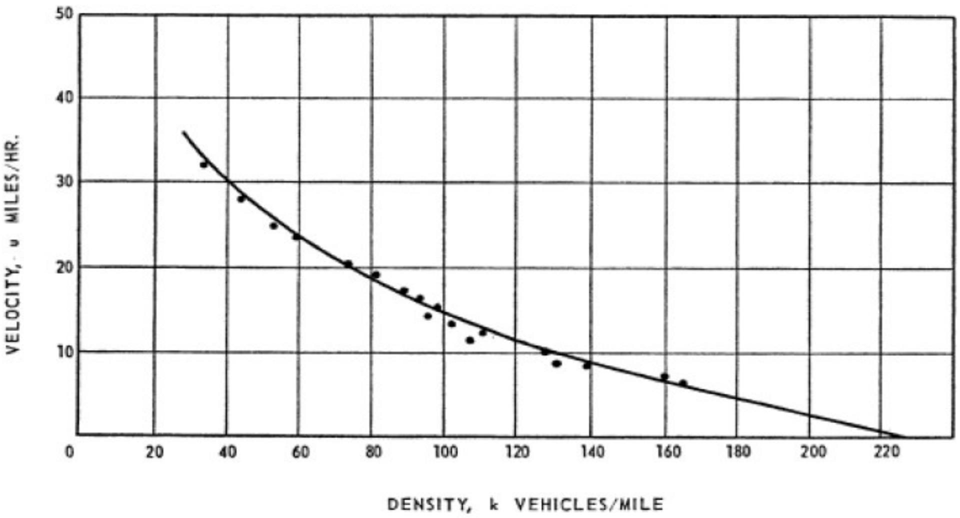


Figure 2.6: Greenberg's speed-density curve based on empirical data [29]

2.6.3 Underwood's model

Since Greenberg's curve was unrealistic at low densities where the speed tends to infinity, Underwood proposed a model which was in the form of an exponential curve where speed was a finite value at zero density [30]. His model however was asymptotic to the X-axis suggesting that even at very high density values the speed does not reach zero. The model proposed by Underwood is shown in equation (2.7),

$$v = v_f e^{-(k/k_c)} \tag{2.7}$$

Where,

v = Average speed (km/h)

v_f = Free flow speed (km/h)

k = Density (veh/km/ln)

k_c = Density at capacity (veh/km/ln)

2.6.4 Pipes-Munjal's model

The Pipes-Munjal model is a model based on Greenshields' model developed by Pipes-Munjal in 1967 [31]. A set of models can be derived by varying the 'n' value in the model as shown in equation (2.8),

$$v = v_f(1 - (k/k_j)^n) \tag{2.8}$$

Where,

v_f = Free flow speed (km/h)

k_j = Jam or maximum density (veh/km)

v = Speed (km/h)

k = Density (veh/km)

n = Real number

2.6.5 Drake's model

Drake et. al developed this model in 1967 based on the FFS and density at capacity [23]. The model developed is shown in equation (2.9),

$$v = v_f . e^{-0.5\left(\frac{k}{k_c}\right)^2} \tag{2.9}$$

Where,

v = Average speed (km/h)

v_f = Free-flow speed (km/h)

k = Density (veh/km/ln)

k_c = Critical density, at which capacity occurs (veh/km/ln)

2.6.6 Maximum Capacity method

The maximum flowrate method is arguably the easiest method to estimate capacity. This is the highest observed flowrate of a location during a given period of time. Here the time interval taken to calculate the flowrate (e.g. 5-min, 15-min, 60-min, etc.) and the total period of flow observation are important factors that affect the estimated capacity [26]. The maximum capacity method equation is shown in (2.10),

$$C_i = \max(f_{i,t}) \quad for\ all\ t = 1,2,\dots,n \tag{2.10}$$

Where,

C_i = Capacity (maximum flowrate) for location i;

$f_{i,t}$ = Observed flowrate in time interval t;

t = Time interval (e.g. 5-min);

n = Number of time intervals considered.

2.6.7 Van Aerde method

Van Aerde [32] put forward a four-parameter model to estimate capacity which has a high number of degrees of freedom to capture a range of behaviours on different facility types. This model has been developed specifically to be calibrated using Intelligent Transportation Systems (ITS) data such as inductive loop data, radar and video detector data [26]. The Van Aerde model is given in equation (2.11),

$$C_i = \frac{u_i}{c_1 + \frac{c_2}{u_{f,i} - u_i} + c_3 u_i} \tag{2.11}$$

Where,

C_i = Estimated capacity for location i;

u_i = Space mean speed (km/h) for location i;

$u_{f,i}$ = FFS (km/h) for location i;

c_1, c_2, c_3 = Headway constant coefficients.

2.6.8 Other capacity estimation methods

Among other capacity estimation methodologies two methods that are quite popular are the Breakdown capacity methodology and the Product Limit Method (PLM). A breakdown occurs when the traffic stream speed decreases past a pre-specified threshold between two consecutive time intervals and is sustained for a predefined time period [33]. By reviewing literature, it was seen that that the pre-defined speed threshold and congestion time are subjective [26]. When considering the product limit method, it is based on the finding by Brilon et al. [34] that capacity based on daily observations of traffic data collected over several months, is Weibull distributed. Based on this finding other researchers have developed the PLM to estimate the capacity distribution function from empirical data. Li & Laurence [26] relate the PLM to the flows that cause breakdown in flow and the flows that do not cause breakdown to develop the capacity distribution function. The issue with these methods and also the Van Aerde method is that they are data intensive methods to estimate capacity. With the limited resources available traffic data of such magnitudes are not easy to collect in Sri Lanka.

As seen above there are a number of empirically derived models that can be utilized in capacity estimation. This creates the question as to which model is most appropriate to be used. The search for the “best” modeling approach is important only if that approach is the best for all cases. But evidence shows that no single model fits best for all situations. Site-specific research studies have demonstrated over time that different models may be the “best” solution for different sites and/or time periods [10].

2.7 Capacity Estimation Studies

This section of the review is primarily focused on countries with heterogeneous traffic conditions. Since observing traffic flows purely formed of passenger cars are not practical under normal circumstances many researchers have developed base capacity values under ideal roadway conditions but with the existing traffic conditions. The vehicle flowrates are usually transformed to passenger car flow rates using equivalency factors as found appropriate.

- Sathishkumar et. al (2016) [35] estimated base capacity of Urban Indian 4-lane roads under ideal roadway conditions. (3.5m lane width, and no roadside friction) The composition of vehicles were 64.8% Cars, 3.7% Heavy vehicles and the rest motor cycles and three wheelers. The lane capacity was estimated to be 1570 pcu/h/ln. The operating speed which was defined as the 85th percentile value of free flow speed of passenger cars was estimated to be 64kmph. In comparison with the existing Indian standard of 900 pcu/h/ln (IRC:106 1990) for Urban 4-lane roads this was found to be a vast improvement.
- Chandra et. al [36] studied the effect of traffic composition on inter-urban multilane highways of India in 2014. Using VISSIM simulation software the lane capacity of a 4-lane highway comprising entirely of passenger cars was found to 2475 pcu/h/ln. The variation of this value with the increase in percentage of other vehicles is shown in Table 2.20, where CB – Cars (Sedans), HV – Heavy Vehicles, 3W – Three Wheelers, and 2W – Motor Cycles.

Table 2.20: Directional capacity change with vehicle composition [36]

Vehicle types	Directional capacity (veh/h) at percent share of second category				
	0 %	10 %	20%	30%	40%
CB	4950	4535	4307	4044	3734
HV	4950	3996	3460	2966	2320
3W	4950	4354	4108	3788	3506
2W	4950	4989	5315	5862	6348

It is observed that the increase in heavy vehicles and three wheelers will decrease the capacity of the road whereas the increase in the percentage of motorcycles would increase the capacity.

- Anamika et al. [37] studied the capacity of inter-urban multi-lane highways in India in 2014 proposing that the capacity per lane on a 4 lane highway is 2250 pcu/h/ln. The capacity value was derived based on the assumption that capacity occurs at half of the free flow speed.

- Yang and Zhang in 2005 [38] found based on model development of traffic flows on multi-lane highways in Beijing that the average roadway capacities on four-lane, six-lane and eight-lane highways are 2104 pcu/h/l, 1973 pcu/h/l and 1848 pcu/h/l respectively. This shows that there is a slight decrease in average lane capacity with increasing number of lanes on a highway.

- In a study done by Madhu and Velmurugan in 2011 [39] the capacity of eight lane Indian expressways with no vehicular segregation were studied incorporating microsimulation software PARAMICS. The lane capacity was derived to be 2859pcu/h/l (speed at capacity approximately 30km/h) when the vehicles were allowed to freely maneuver within road space. When lane following rules were applied the capacity reduced to 2449 pcu/h/l. This was further reduced to 2235pcu/h/l when no lane changing was allowed.

- In a study done by Semeida in 2013 [40] on rural multi-lane highways in Egypt, the capacity, LOS and the factors affecting capacity where investigated. 45 highway sections were studied, and LOS and capacity were estimated using the HCM 2000 methodology. Capacity values between 1477 pcu/h/l and 2200 pcu/h/l were observed on these sections. The study used lane width, directional pavement width, lateral clearance, number of lanes per direction, median width, side access availability, percentage of heavy vehicles in traffic stream as independent variables to estimate capacity. The 45 highway sections were categorized into two as Desert road sections and Agricultural road sections. These road types can be loosely defined as follows: Desert road sections are those with minimum roadside developments (Rural roads) whereas agricultural road sections are those that pass through sub-urban areas. The developed models for all sections, agricultural roads, and desert roads are shown in equations (2.12), (2.13), (2.14) respectively.

$$Capacity_{ALL} = 818.17 - 358.2(SA) + 371.01(LW) \tag{2.12}$$

$$Capacity_{Agr.} = 1960.98 - 270.8(SA) + 76.1(LC) \tag{2.13}$$

$$Capacity_{Desert} = 1199.7 + 101.81(LC) + 181.98\sqrt{MW} - 6002.73(HV^2) \tag{2.14}$$

Where, SA is side access availability (input: 0,1), LW is lane width in meters, LC is lateral clearance in meters, MW is median width in meters, and HV is the heavy vehicle percentage.

2.8 Capacity Adjustment factors

2.8.1 Lane width

The HCM 2010 defines lane widths starting from 3m with 3.6m being the ideal lane width. Any lane with width greater than 3.6m is said to operate with the same characteristics with that of a 3.6m lane. This is a satisfactory assumption given the lane discipline and car following behaviour observed in developed countries. According to the HCM 2010 guideline a lane with width between 3.0m - 3.3m would reduce the capacity by 100pcu/h/ln given that all other factors are constant. Which is a reduction of approximately 5% of the capacity.

The Indonesian HCM suggests adjustment factors for lane width varying between 3m and 4m with the standard value being 3.5m for both divided and undivided urban four lane highways. The reduction in base capacity for divided and undivided four lane highways are 8% and 9% respectively when the lane width is 3m. Similarly, the increase in capacity is 8% and 9% when the width is increased to 4m.

<u>Studies done on the variation of capacity with lane width is documented below</u>

Chandra and Kumar [41] examined the influence lane width has on two-lane roads in India in 2003. They developed a model relating the carriageway width (w) with capacity (C) as shown in equation (2.15),

$$C = -2184 - 22.6w^2 + 857.4w \tag{2.15}$$

Roads with carriageway width varying from 5.5m to 8.8m had been surveyed in this study. This goes past the 7.2m cap given for two-lane roads in the HCM. This can be understood given the fact that traffic streams of heterogeneous nature will use whatever space available for movement.

In 1968, a researcher Leong [42] measured the capacity of 31 rural two-lane highways in Australia with the objective of finding the influence lane width has on capacity. The results showed that the capacity of a two-lane road can drop by a margin of 28% when the lane width is decreased from 3.70 m to 2.75 m.

Nakamura [43] studied the highway capacity on Japanese roads in 1994, and suggested an adjustment factor (YL) for lane widths (WL) less than 3.25 m in the model shown in equation (2.16). This factor was common for both two-lane and multi-lane highways.

$$YL = 0.24WL + 0.22 \tag{2.16}$$

2.8.2 Median type

The HCM 2010 manual defines three median types for a multi-lane highway; namely, Divided, Undivided, and Two way left turn lane (TWLTL). Divided and TWLTL are ideal conditions whereas undivided lanes are said to affect the FFS adversely, reducing the capacity of the segment. The reduction in FFS due to this factor is 1.6 mi/h (2.5km/h) which is unlikely to cause a major reduction in capacity.

The Indonesian HCM defines two states, Divided and Undivided, with adjustment factors according to the directional split of vehicles travelling each direction in an undivided road. The capacity can be reduced up to 6% from its original value for four lane undivided roads (see Table 2.13).

Other research on the effect of the presence or absence of the median on capacity is sparse. Moses and Mtoi [44] pointed out that the presence of a median has a statistically significant positive effect on FFS on urban arterials in a report published in 2013.

2.8.3 Free Flow Speed (FFS)

The HCM 1994 defined capacity in relation to FFS. So did the subsequent manuals in 2000 and 2010. The HCM 2010 defined 4 FFS values of 97km/h, 89km/h, 80km/h, 72km/h with their respective capacities being 2200, 2100, 2000, 1900 pcu/h/l (see

Table 2.6). Interpolation between FFS curves was discouraged by providing a range of FFS values per speed-flow curve. The reduction in 5mi/h (8km/h) in FFS would reduce the capacity approximately 5%.

Other research studies,

- Arun et. al [45] studied the variation of capacity with FFS on Indian inter-urban highways in 2016.

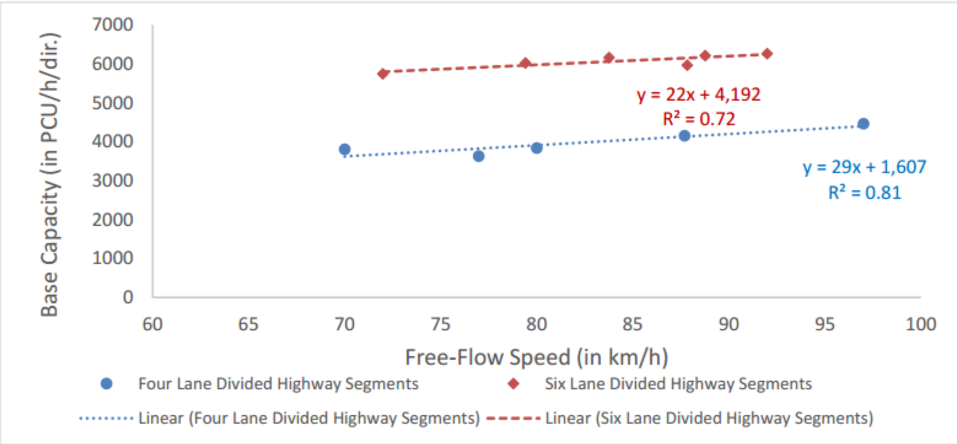


Figure 2.7: Base capacity vs. FFS [45]

As shown in Figure 2.7 Figure 2.7: Base capacity vs. FFS both six-lane and four-lane highways show an increase in Capacity with the increase in FFS. The four-lane highway shows an increase of approximately 100 pcu/h/ln which is equal to what the HCM 2010 suggests.

- Satishkumar et. al [35] studied the relationship between operating speed of passenger cars with capacity of urban four-lane divided roads in 2016. Operating speeds where defined as 85th percentile speeds of the FFS. The developed model is shown in equation (2.17).

$$Lane\ capacity\ =\ 14.222(operating\ speed)\ +\ 1001.4 \qquad (2.17)$$

2.8.4 Access Point Density

The HCM 2010 defines a reduction factor for access point density as shown in Table 2.10. HCM states that for each access point per mile, the FFS is reduced by approximately 0.25 mi/h (4km/h), regardless of the presence of the center median.

Further it states that the inclusion of a given access point is subjected to the engineer's judgement.

The Indonesian HCM tackles the issue of access points along with some other friction factors encountered on roadsides in Asian countries. It defines a factor called "Side Friction Class" which is shown in Table 2.15. The side friction class can be determined by aggregating the incidents according to the given guidelines. But this factor is not directly applied as an adjustment to capacity. It is further combined with the shoulder/curb width of the given road.

Chand et. al [46] studied the drop in capacity due to curb-side bus stops in India in 2014 and concluded that there is a drop of 8-13% in base capacity due to this phenomenon.

Pallavi et. al [47] investigated the effect of side friction versus stream speed in urban multilane mid-block sections by categorizing side friction classes according to the IHCM 1997 guideline. They concluded that the low to medium side friction classes do not have a significant effect on stream speed whereas at sections with high side friction classes the reduction in speed was significant.

2.8.5 City Size

The IHCM 1997 defines a factor with respect to the number of inhabitants of a city. This factor is said to portray the development of a city and hence the average performance of vehicles in the traffic stream (composition of traffic and engine performance). (See Table 2.17)

2.8.6 Shoulder type and width

The HCM 2010 methodology defines shoulder width in terms of lateral clearance, where the total vacant width on either side of a road is considered. The extent of free space on either side of the road is said to have a positive or freeing effect on the drivers.

The base value is 3.6m in total, with 1.8m on either side. A four-lane highway with no lateral clearance will have approximately 5% reduction in capacity while the effect on a six-lane highway is slightly less (see Table 2.8).

The IHCM 1997 incorporates the shoulder type and width to define its adjustment factors in the side friction factor parameter. This is more representative of roadside conditions of Asian countries. Two categories, shoulder and curb with distinct adjustment factors are defined. The ideal condition is a function of the side friction class and the effective width (see Table 2.15).

In other research, Leong (1978) measured speeds and capacity values on rural highway sections with varying lane width and shoulder width in New South Wales. Analysis of the data suggested that speed increased with the increase in shoulder width. [48]. Prakash (1970) also stated that capacity is considerably influenced by the type and width of shoulder present [49]. The consensus on shoulder and lateral clearance is that the increase in width has a positive effect on roadway capacity.

2.8.7 Vehicle Composition

The HCM 2010 does not incorporate the vehicle composition in its estimation of capacity. The effect of heavy vehicles is used in the level of service calculation. Neither does the IHCM 1997 directly incorporate vehicle composition in capacity estimation. It however indirectly addresses this issue through the city size adjustment factor in which the vehicle composition plays a role (Table 2.21).

Table 2.21: Vehicle composition with city size in IHCM 1997 [27]

City size	Light Vehicle (LV) %	Motorcycle (MC) %	Heavy Vehicle (HV) %
<0.1 million inhabitants	45	10	45
0.1-0.5 million inhabitants	45	10	45
0.5-1.0 million inhabitants	53	9	38
1.0-3.0 million inhabitants	60	8	32
>3.0 million inhabitants	69	7	24

Chandra et. al [36] studied the variation of capacity with vehicle composition on Indian multi-lane roads with the help of VISSIM simulation software in 2015. It was discovered that capacity of the highway decreases when the percentage of vehicle types: CB (Car-Big), HV (Heavy Vehicle) and 3W (Three-wheeler) increases in the traffic stream. On the other hand, it was observed that when the percentage of 2W (Motorcycle) increase in the traffic stream the capacity increases (Table 2.22).

Table 2.22: Capacity change with vehicle type [36]

Vehicle types	Directional capacity (veh/h) at percent share of second category				
	0 %	10 %	20%	30%	40%
CB	4950	4535	4307	4044	3734
HV	4950	3996	3460	2966	2320
3W	4950	4354	4108	3788	3506
2W	4950	4989	5315	5862	6348

The issue of different vehicle classes operating in the same traffic stream is negated by incorporating the 'Passenger Car Unit' (PCU) or 'Passenger Car Equivalent' (PCE) to bring the unit of flow to a common denominator. Hence direct comparisons of vehicle composition and capacity are rare. Further, as seen in section 2.6 vehicle composition is not taken into account by any of the capacity estimation methodologies currently in use. This is tackled by having different capacity values for different regions as the vehicle composition is generally native to the country/region.

2.8.8 On-street Parking

The IHCM 1997 touches on on-street parking when deriving the ‘side friction class’ adjustment factor for urban roads. The HCM does not provide a methodology to incorporate this effect in capacity calculations. In a study done in Australia in 2015, Wijerathna concluded that half hour on-street parking zones reduced the theoretical capacity of urban roads by a percentage up to 17%. [50].

2.9 Passenger Car Unit (PCU) Factor

Passenger Car Unit (PCU), also referred to as the Passenger Car Equivalent (PCE), is a metric used in transportation studies to measure the traffic flowrate on highways. PCU is essentially the impact a given mode of transportation has on traffic parameters (e.g.: speed, density, headway) in comparison to that of a passenger car. Numerous studies have been done worldwide on PCU factors and different methods have been proposed by researchers to estimate PCU values.

2.9.1 PCU Estimation methods

PCU estimation methods can be categorized based on the characteristics they are founded upon such as flowrate, density, speed, etc.

<u>Based on Flowrates and Density</u>

John & Glauz, (1976) proposed the following equation (2.18) based on the truck volume to capacity ratio, mixed vehicle flow and grade [51].

$$PCE = \frac{q_B - q_M(1 - P_T)}{q_M x P_T} \quad (2.18)$$

Where,

q_B – Equivalent passenger car flow rate for a given v/c ratio

q_M – Mixed flow rate

P_T – Truck proportion in the mixed traffic flow

Huber, (1982) proposed a model (equation (2.19)) to estimate PCU under free flow conditions, relating the ratio between volumes of two streams at some common level of impedance [52].

$$PCE = \frac{1}{P_T}\left(\frac{q_B}{q_M} - 1\right) + 1 \quad (2.19)$$

Where,

q_B – Equivalent passenger car flow rate for a given v/c ratio

q_M – Mixed flow rate

P_T – Truck proportion in the mixed traffic flow

The modified density method [53] for PCU calculation is another method developed based on the density method used in HCM 2000 for heterogeneous traffic conditions. This method has some shortcomings such as high sensitivity to the density forecast of cars and being a complex procedure.

Based on Headways

Greenshields, et al., (1947) proposed the *basic headway method* denoted by equation (2.20) [54].

$$PCU_i = {}^{H_i}/_{H_c} \quad (2.20)$$

Where,

PCU_i – Passenger Car Unit of ith vehicle type

H_i – Average headway of ith vehicle type

H_c – Average headway of passenger car

Krammes and Crowley (1986) developed a method based on the factors that contribute to the overall effect of trucks on the roadway type. The model is denoted by equation (2.21) [55].

$$PCE = [(1 - P_T)H_{TP} + pH_{TT}]/H_P \tag{2.21}$$

Where,

H_{TP} – is the lagging headway of trucks following passenger cars,

H_{TT} – is the lagging headway of trucks following trucks,

H_P – is the lagging headway of cars following either vehicle type

Based on Speed

Van Aerde and Yagar (1983), developed a method to estimate PCE based on the relative speeds of each type of vehicle traveling in the main direction and for the collective of vehicles traveling in the opposite direction. The linear regression model structure is as given by equation (2.22) [56].

$$\begin{aligned} \mathit{Percentile\ speed} &= \mathit{free\ speed} + C_1(\mathit{number\ of\ passenger\ cars}) \\ &+ C_2(\mathit{number\ of\ passenger\ trucks}) + C_3(\mathit{number\ of\ RVs}) \\ &+ C_4(\mathit{number\ of\ other\ vehicles}) \\ &+ C_5(\mathit{number\ of\ opposing\ vehicles}) \end{aligned}$$

Where C_1 to C_5 are Speed reduction coefficients for each vehicle type.

$$E_n = C_n/C_1 \tag{2.22}$$

Where,

E_n – PCE factor n vehicle type,

C_n – Speed reduction coefficient of n vehicle type,

C_1 – Speed reduction coefficient of passenger cars

Chandra et al., (1995) developed a model to calculate PCU for mixed traffic conditions [57]. The PCU is a function of the speed and projected area of the vehicle as shown in equation (2.23).

$$PCU_i = \frac{V_{car}/V_i}{A_{car}/A_i} \tag{2.23}$$

Whrere,

PCU_i – Passenger car unit value of i^{th} vehicle type

V_{car} – Speed of car (km/h)

V_i – Speed of i^{th} vehicle type (km/h)

A_{car} – Static (projected rectangular) area of the passenger car (m^2)

A_i – Static (projected rectangular) area of the i^{th} vehicle type (m^2)

Based on Delays

Craus et al. (1980) in their equivalent delay method developed a model based on the difference between the delay caused by a heavy vehicle to a standard car and delay caused by a slower car to standard car [58]. The model is denoted by equation (2.24).

$$PCE = d_{kt}/d_{kp} \tag{2.24}$$

Where,

d_{kt} – Average delay time caused by one truck,

d_{kp} – Average delay time caused by one passenger car.

Cunagin and Messer (1983) developed a PCE estimation based on speed distribution, traffic volumes, and vehicle types [59]. The PCE values were determined by using spatial-headway and equivalent-delay methods. The model is shown in equation (22).

$$PCE = \frac{D_{ij} - D_{base}}{D_{base}} \tag{22}$$

Where,

D_{ij} – Delay to passenger cars due to vehicle type i under conditions j,

D_{base} – Delay to standard passenger cars due to slower passenger cars

The majority of the PCU estimation methods are developed based on homogeneous traffic conditions for homogeneous traffic streams. Of the PCU estimation methods discussed above Chandra's method and the Modified Density method are based on heterogeneous traffic conditions.

2.9.2 Sri Lankan PCU Studies

In one of the most prominent studies in the area of PCU locally done by Kumarage in 1996, highway sections, roundabouts and signalized intersections where examined and PCU factors for each type were calculated [60]. The PCU values for highway sections are calculated using the Speed-Flow method, which is an indirect method of determining PCU values by using the speed-flow relationship. Table 2.23 shows the values developed in the study.

Table 2.23: PCU values developed by Kumarage (1996)

Highways (Plain & rolling terrain)	Cart	Bicycle	Motorcycle	Three-Wheeler	Car, Van	Minibus, Bus	Light truck, Medium Truck	Large Truck
Single Lane	6.1	0.5	0.2	0.6	1	2.6	2.1	5.7
Interim Lane	-	-	0.4	-	1	1.8	1.3	-
Two-Lane Undivided	2.5	0.7	0.4	0.8	1	1.8	1.5	3
Four Lane Divided	3.7	1	0.6	0.9	1	1.7	1.5	4

A more recent study was carried out by Weerasinghe and Pasindu in 2015 for four-lane roads in Sri Lanka [61]. The methods adopted to calculate PCU was Chandra's method and the modified density method. Table 2.24 indicates the results obtained by both methods.

Table 2.24: Comparison of PCU values developed by Weerasinghe and Pasindu [61]

Type	PCU factor value					
	Motorcycles	Three-Wheelers	Car	Van	Light Bus	Heavy Vehicles
Modified Density Method	0.89	1.02	1	1.45	2.99	2.3
Chandra's method	0.3	0.74	1	1.39	4.89	4.21

Considering the two methods (Chandra's method and Modified Density method) suited for heterogeneous traffic conditions it is seen that Chandra's method is more suitable for PCU calculations given its wide use in literature [3], [4], [6], [33]-[34], [41], [57]-[60] simple model and consistent output of results. Further, in a recent study done by Raj, Asaithambi and Ravi Shankar in 2019, they concluded that Chandra's method was most suited for PCU estimations on highway midblock sections [62]. This study compares Chandra's method and the Modified Density method with a number of other PCU estimation methods under heterogeneous traffic conditions.

2.10 Data Collection methods

Depending on the data and accuracy of data required, different methods can be adopted to collect traffic data. Traffic data can broadly be categorized into two types, Macroscopic data and Microscopic data. Macroscopic traffic data deal with traffic stream data such as, traffic stream speed, traffic flow, density etc. whereas microscopic traffic data deal with vehicle to vehicle interactions and individual vehicle data such as speed, lateral position, trajectory, acceleration etc. Both types of data are important in the study of capacity as capacity models can be built based on both types of data.

Following are some of the methods that are used to collect traffic data,

1. Manual data collection
2. Video based methods
3. Radar based methods
4. Pressure Contact Tubes
5. Google traffic data

2.10.1 Manual Data Collection methods

Manual vehicle counts also known as Manual Classified Counts (MCC) is a method of flow data collection by observing vehicles on the road. This is usually done by employing enumerators to collect the necessary types of traffic data as per the requirement. This method is still useful as at present automated methods do not accurately gather some data types such as vehicle occupancy, vehicle classification, pedestrian details etc. [65]. Even though it's possible to collect data types such as speeds of individual vehicles by employing techniques such as the 'number plate method' these surveys are resource intensive. Hence the at present manual data collection methods are limited to flow data collection.

2.10.2 Video based methods

This method involves the collection of data using video cameras. A well-placed video camera can be used to capture traffic along a significant length of the road. If the distance between two points in the captured video is known the speeds of individual vehicles can be calculated. Many studies have used the simple videography technique where the videos are manually analyzed later, to collect traffic data including a study done by Jayaratne et. al [3] in 2016 to calculate vehicle speeds on two-lane roads in Sri Lanka [66, 67, 68, 69]. Most traffic data types can be collected using this method. Further, it has the added advantage of having visual proof of collected data for later reference. Hence this is one the most used data collection methods in the field of traffic engineering.

The cost incurred in the videography method is low compared to the other methods and it reduces the manpower requirement. But extraction and analysis of traffic data from a video is a tedious process. Hence software programs such as TRAIS, COUNTcam, TrafficVision, TRAZER, MediaTD, Picomixer STA etc. have been developed to automate this process. Image processing techniques are used mainly in these software applications and manual verification is generally made possible [70].

2.10.3 Radar based methods

Radar based methods include variety of instruments from handheld speed guns to stationary infrared (IR) data collection devices. The principle behind IR systems is the intervention of Infra-Red beams. When a vehicle passes by and obstructs the Infra-Red rays, it detects and counts the vehicle. This method has a vast number of capabilities based on how the technology is used including the ability to measure the speed, length and lateral placement of a vehicle. The Infra-Red Traffic Logger (TIRTL) is a traffic data collection system which uses this technology. One of the drawbacks in this method is there being no visual backup of the collected data for cross reference. Shou, Yingzi & Yi (2010) compared the classification of vehicles of TIRTL instrument under different weather conditions. It was observed that in clear weather conditions, fog, snow and rain the TIRTL vehicle counts agreed very well with the actual counts. During thunderstorms the TIRTL instrument undercounted the number of vehicles [71].

2.10.4 Pressure Contact Tubes

Pressure contact tubes (pneumatic tubes) can be used similar to infrared counters. Data such as vehicle classification, speed and flow can be collected using this method. This is used by the Road Development Authority (RDA) in Sri Lanka for traffic flow data collection.

2.10.5 Google Distance Matrix (GDM) method

The Google Distance Matrix Application Programming Interface (API) is a service that provides travel distances and travel times for origin and destination pairs. The API returns information based on the given route between origin and destination points, as calculated by the Google Maps API. This feature can be used for traffic stream speed estimations of road segments of varying length. A study carried out by Kumarage estimated that the travel time can be predicted using Google Distance matrix API data to an accuracy of up to 99% [72]. The same methodology can be extended to predict traffic stream speeds of road links.

2.11 Summary of Literature Review

The traffic carrying capacity of roads was initially discussed in the early 1930's and has been evolving ever since. Capacity was initially tied with the onset of congestion by AN Johnson before later being refined to the current definition provided by the 2010 HCM. An important study was carried out by Greenshields' in the early 1930's where he developed a relationship between speed and flow which is still in use.
This chapter discusses the evolution of the definitions and values of capacity given in the HCM manuals from 1950 up until 2010. The value of capacity was initially declared to vary between 1000-2000 pcu/h/ln in the 1950 manual and is currently refined to a value between 1900-2200 pcu/h/l in the 2010 HCM. Next the factors that affect capacity as given in the HCM guidelines were studied. While factors such as the lane width and lateral clearance were identified early on, factors such as median type, access point density were added in later iterations of the manual.

The HCM 2010 multi-lane capacity estimation methodology is discussed next. The methodology's limitations, base conditions and its framework to establish capacity is examined. It was observed that capacity is defined based on the FFS which is dependent upon the lane width, median type, access point density and lateral clearance. Next the Indonesian HCM (IHCM) capacity estimation methodology is examined. This methodology is based on the 1994 HCM capacity model and has been adjusted to suit Indonesian road conditions. The manual states that multi-lane capacity is dependent upon the lane width, directional split, side friction, shoulder type and width, and the city size. According to the IHCM the multi-lane capacity may vary between 850 pcu/h/l and 1900 pcu/h/l. The local, RDA guideline which is based on the 1985 HCM methodology defines capacity to be 2000 pcu/h/l with no adjustment factors.

Next the literature available on capacity adjustment factors are reviewed. In this section it was observed that lane width, median type, FFS, access point density, city size, shoulder type and width (lateral clearance), vehicle composition, and on-street parking were the major factors that influenced capacity. Empirical models available for capacity estimation and PCU estimation were reviewed in this chapter. It was observed that different models required different levels of data. Hence which type of model is most suited for the study was analyzed. When considering the PCU estimation methods it was observed that some methods were more suited for heterogeneous traffic conditions, hence those were reviewed with the intention of using the best method for this study. Finally, the different traffic data collection methods available were discussed and compared in order to ascertain which method or methods would most suit the requirements of this study.

3 RESEARCH PILOT STUDIES

3.1 Testing HCM 2010 Applicability to Sri Lanka

The Highway Capacity Manual 2010 is the most widely used guideline for capacity estimations. The multi-lane capacity estimation methodology of this manual was reviewed in section 2.2. Due to factors discussed in section 2.2.1 it is unlikely that the HCM 2010 methodology will produce accurate estimations of capacity for Sri Lankan roads. This hypothesis is tested in this section of the thesis.

The capacity of a multi-lane highway is dictated by the FFS in the HCM 2010. Hence, to estimate the capacity of a section the FFS of that section needs to be found. As explained in section 2.2.3.1 the FFS can be derived in one of two ways; either through field measurement or by using the FFS model given in the manual. (Equation (2.1)).

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A \tag{2.1}$$

Where, FFS is the free flow speed, BFFS the base free flow speed, f_{LW} the lane width adjustment, f_{LC} the lateral clearance adjustment, f_M the median type adjustment and f_A is the access point density adjustment.

Five multi-lane sections were chosen for the study and the relevant geometric details of the sections were collected and are as shown in Table 3.1. The Google Distance matrix (GDM) method and equation (2.1) both were used to calculate the Free Flow Speeds at each location. The FFS values calculated from equation (2.1) (as explained in section 2.2.3.1) based on the geometric details of the highway section, are shown in the last column of Table 3.1.

Table 3.1: Geometric details of sections considered

Location	Posted Speed (mi/h)	Lane width range (ft)	Lateral Clearance (ft)	Median Type	Access Point Density (per mile)	FFS (mi/h)
A1 Highway Loc 1	31 (50km/h)	11-12 (3.35-3.65m)	4 (1.2m)	Divided	0 (0 per km)	34 (55 km/h)
A1 Highway Loc 2	31 (50km/h)	11-12 (3.35-3.65m)	8 (2.4m)	Divided	13 (8 per km)	32 (52 km/h)
A3 Highway	31 (50km/h)	11-12 (3.35-3.65m)	8 (2.4m)	Divided	23 (14 per km)	30 (48 km/h)
Marine Drive	31 (50km/h)	11-12 (3.35-3.65m)	6 (1.8m)	Undivided	24 (15 per km)	27 (44 km/h)
New Panadura Road	43.5 (70km/h)	11-12 (3.35-3.65m)	12 (3.6m)	Divided	23 (14 per km)	42.5 (69km/h)
A4 Highway	31 (50km/h)	11-12 (3.35-3.65m)	4 (1.2m)	Undivided	9 (6 per km)	32 (52km/h)

24-hour speed data were collected across 25 days at each of the 5 locations at 10-minute intervals using the GDM method. The 95th percentile speed obtained from this method was considered to be equivalent to the FFS. The speed statistics of the data collected using the GDM method are shown in Table 3.2.

Table 3.2: Collected speed statistics for study

Location	Top Speed (km/h)	95th Percentile Speed (km/h)	16-hour top speed (km/h)	Min speed (km/h)	No. of Data Points
Kelani Bridge (A1)	58	52	52	5	3569
A1	50	48	48	19	3578
A3	47	43	45	9	3579
Marine Drive	45	42	43	7	3580
New Panadura Road	64	60	60	11	3580
Pannipitiya (A4)	64	55	59	2	3580

A comparison of the FFS from the two methods is shown in Figure 3.1.

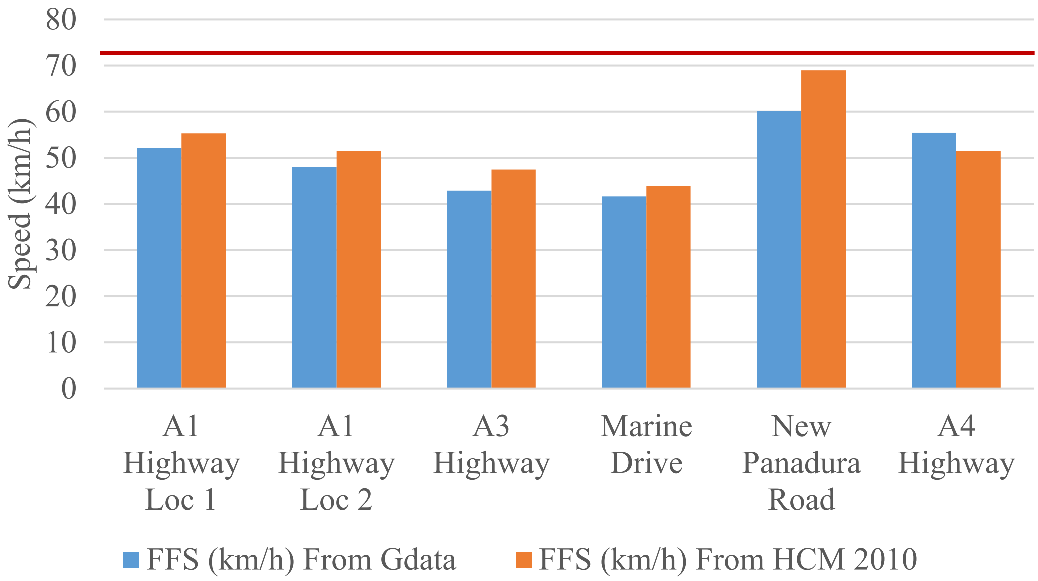


Figure 3.1: Comparison of GDM data speeds and FFS from equation (2.1)

By observing the results, it is seen that HCM 2010 FFS model accurately estimates the actual FFS. The average deviation between the two speed values is 4.9km/h (10%). However, the issue is with the FFS values observed. They are considerably lower than what is expected in the HCM 2010 methodology. The maximum FFS value from the study of the five highway sections was 69km/h (average observed FFS 50km/h). But the HCM 2010 defines capacity values for FFS values greater than 72km/h (marked in red in Figure 3.1). Hence based on the FFS, the HCM 2010 multi-lane capacity methodology is not applicable to Sri Lankan roads. Further, it should be noted that there are other limitations (discussed in section 2.2.3.1) in applying the methodology to Sri Lanka excluding the FFS.

3.2 Capacity Evaluation - Pilot Study

The approach employed to estimate capacity in this study was through curve fitting. A pilot study was carried out to test out this method. Figure 3.2 shows the capacity estimation methodology workflow chart.

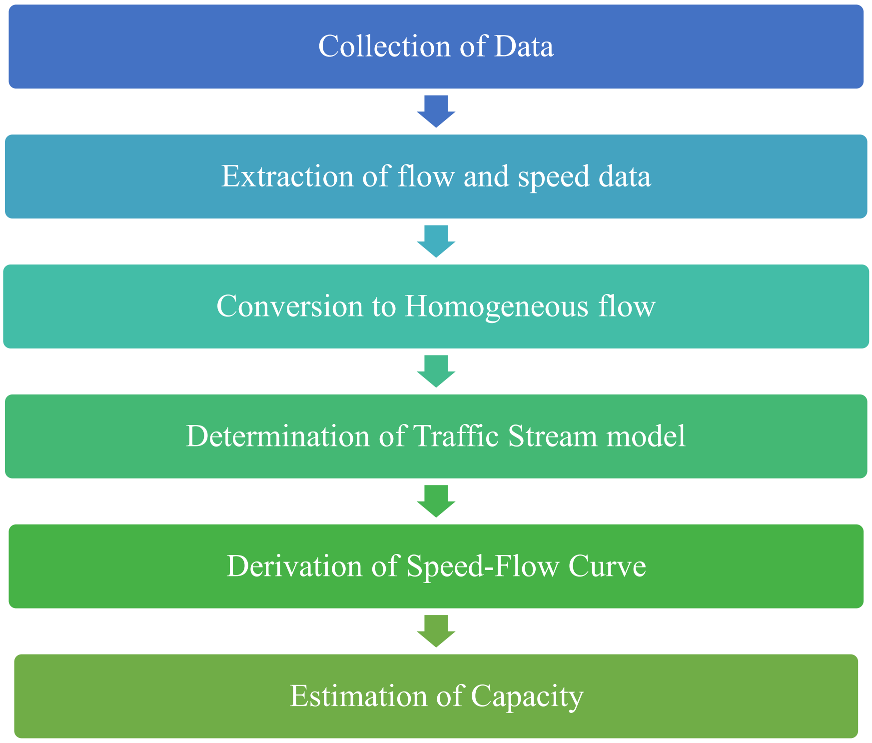


Figure 3.2: Capacity estimation methodology

3.2.1 Data collection of Pilot Study

Data collection for this study was done by the method of video recording. 12-hour video clips were recorded between the time period 0800 – 2000hrs at 3 locations to gather the traffic data. The video camera was placed in such a way that there was no interference to the vehicles and both directions of the road section are captured. The road sections selected were on A01 and A03 highways as shown in Figure 3.3. The geometric details of the roads are as follows,

- Number of lanes per direction – 2 lanes
- Lane width – 3.5m
- Access control – Minimum access

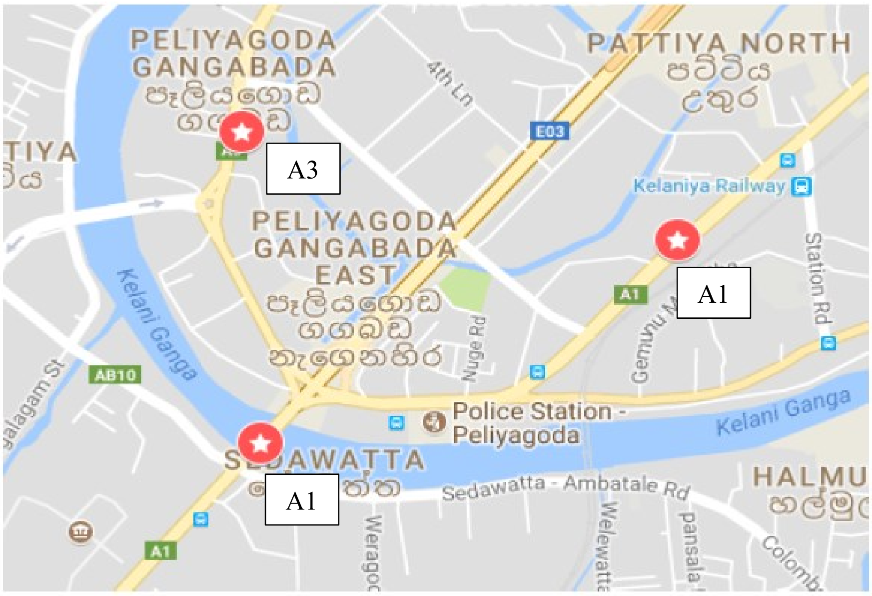


Figure 3.3: Data collection points for study

3.2.2 Extraction method of Flow and Speed data

5-minute interval flow counts were extracted from the video to create a database of flow values varying with time. The vehicle flow counts of each lane was counted separately. The flow counts were classified by vehicle into 11 categories. Namely, Motorcycle (MC), Three-Wheeler (TW), Car, Van, Utility Vehicle, Mini Truck, Medium Truck, Truck, Multi axle, Minibus, and Large Bus. A total of over 27,000 vehicles are were analyzed during this study. The vehicle composition observed during the survey is shown in Figure 3.4. The dominant vehicle category was the MC with share of 29%. Overall, 51% of the traffic stream was made up of small vehicles (MC and TW). The percentage of passenger cars was 24%. This shows the heterogeneity of the traffic stream.

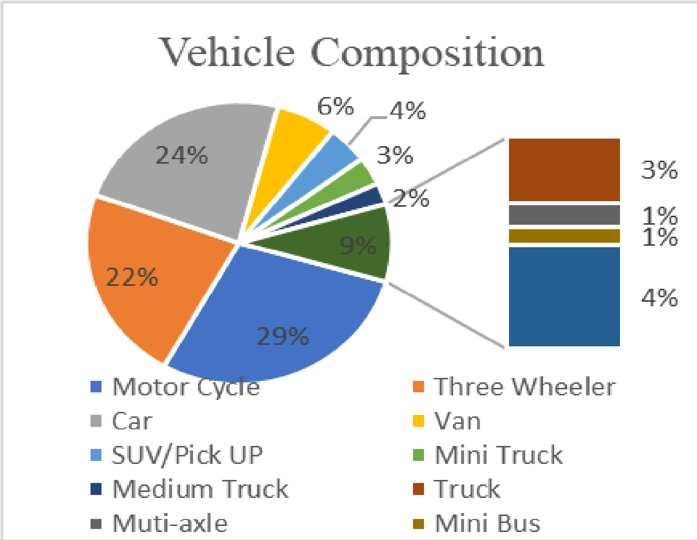


Figure 3.4: Vehicle composition observed in study

Speed counts were carried out for each 5-minute interval to get the speeds of each vehicle category. The space mean speeds (km/h) were calculated by measuring the time for each vehicle to cross a predefined trap length observed in the video. As different vehicle categories have different speed ranges, the result obtained from random sampling may be misleading and thus stratified random sampling based on the volume proportion of different modes was used. To ensure a representative sample, a minimum of 10% from the total volume of each mode observed in 5-minute interval was considered for sampling and finally mean space mean speed was calculated. The vehicle arriving at every 10th second irrespective of mode and lane was considered for speed calculation and the time taken to traverse the considered section was noted down until required sample size was achieved. If the required sample for a particular mode was completed, then that mode is excluded, and the remaining mode was considered for speed calculation. If the extracted sample was less than the required sample size, then vehicles arriving at every 5th second was considered for data extraction and the process was continued until completion.

3.2.3 Method of conversion to Homogeneous flow

The flows observed in the study and in general in Sri Lanka are heterogeneous traffic flows. Hence this needs to be converted to a homogeneous traffic flow for the ease of analysis. The tool used for this conversion is the PCU factor. As discussed in section 2.9 there are many PCU methods available to convert flows to a single unit. From the literature review it was established that Chandra's method (equation (2.23)) was the most suited PCU method to convert heterogeneous traffic flows to a homogeneous traffic flow.

$$PCU_i = \frac{V_{car}/V_i}{A_{car}/A_i} \tag{2.23}$$

Where,

PCU_i – Passenger car unit value of i^{th} vehicle type

V_{car} – Speed of car (km/h)

V_i – Speed of i^{th} vehicle type (km/h)

A_{car} – Static (projected rectangular) area of the passenger car (m^2)

A_i – Static (projected rectangular) area of the i^{th} vehicle type (m^2)

The projected areas (A_i) of the different vehicle categories were taken from a study done by Jayaratne et. al [3]. In this Jayaratne et. al studied over 9,000 vehicles in the traffic stream and based on the percentage of different vehicle makes in each category a weighted projected area was calculated. The projected areas used is shown in Table 3.3.

Table 3.3: Vehicle projected areas

Vehicle Category	**Projected Area (m^2)**	A_{car}/A_i
Car	6.77	1.00
Van	7.96	0.85
Motorcycle	1.54	4.40
Three-Wheeler	3.07	2.21
Utility Vehicle	9.89	0.68
Mini Truck	5.70	1.19
Medium Truck	13.85	0.49
Large Truck	18.79	0.36
Multi Axle	33.46	0.20
Minibus	15.54	0.44
Large Bus	29.84	0.23

Hence using equation (2.21) the PCU values for each vehicle category was calculated for each 5-min time interval using the average speed of vehicle type i and the average speed of the car in that time interval. The vehicle speed information is shown in Table 3.4. The average PCU values of the study is shown in Table 3.5 along with PCU values of other published local research studies. It is seen that the average PCU values derived in this study are comparable with those published in other research. However, there is a considerable disparity between the PCU values given by the RDA and other research. This could be due to the fact that those values are based on a study done back in 1996 where the vehicle interactions in the traffic stream were different than what is observed at present.

Table 3.4: Speed statistics of vehicles surveyed

Vehicle Category	Speed (km/h)		
	Average	Minimum	Maximum
Car	44.7	7.7	75.5
Van	43.6	6.6	69.8
Motorcycle	43.7	9.1	61.5
Three-Wheeler	35.3	6.9	54.0
Utility Vehicle	44.1	6.8	72.9
Mini Truck	40.5	5.2	73.4
Medium Truck	36.1	6.8	64.2
Large Truck	34.2	5.7	60.6
Multi Axle Vehicle	32.9	4.5	58.8
Minibus	40.8	8.7	75.1
Large Bus	39.6	6.9	67.8

Table 3.5: Comparison of derived PCU factors with PCU factors in literature

Vehicle Category	PCU factors			
	This study	RDA values	Jayaratne et. al (2016)	Weerasinghe & Pasindu (2015) [61]
Car	**1.0**	1.0	1.0	1.0
Van	**1.2**	-	1.2	1.39
Motorcycle	**0.2**	0.5	0.2	0.30
Three-Wheeler	**0.6**	0.8	0.6	0.74
Utility Vehicle	**1.7**	-	1.4	-
Mini Truck	**1.0**	1.5	1.2	-
Medium Truck	**2.5**	1.7	-	-
Large Truck	**3.7**	2.8	3.2	4.21
Multi Axle	**6.5**	4.0	-	-
Minibus	**2.3**	1.6	-	-
Large Bus	**5.4**	1.8	4.1	4.89

The heterogeneous flows (veh/5-min/l) were converted to homogeneous flowrates (PCU/h/l) using the PCU factors using equation (3.1),

$$Flowrate\ = \Sigma\ Count\ of\ i^{th}\ vehicle\ category\ x\ PCU_i\ x\ 12 \qquad (3.1)$$

An example of how this is done is shown in Table 3.6.

Table 3.6: Flowrate calculation example

Vehicle category	Car	Van	MC	TW	Utility V.	Mini Truck	Medium Truck	Large Truck	Multi Axle	Minibus	Large Bus	Total
5min Flow	25	13	26	21	4	3	5	3	1	4	6	111
PCU factor	1	1.2	0.2	0.6	1.7	1	2.5	3.7	6.5	2.3	5.4	
PCU flow	25	15.6	5.2	12.6	6.8	3	12.5	11.1	6.5	9.2	32.4	139.9
PCU flow rate	= 139.9 x 12 (PCU/h/l)											1679

3.2.4 Determination of Traffic Stream model (Curve fitting)

Since the speed-flow relationship follows a complex form, the speed-density relationship is used for curve fitting. For this purpose, the corresponding density values of the measured speed and flow values were required. The fundamental flow equation (2.4) was used to derive the density values. Hence, for each 5-min speed (km/h) and flowrate (PCU/h/l) the corresponding density (PCU/km/l) was calculated. A sample of the data set is shown in Table 3.7.

$$Q = U \: x \: K \tag{2.4}$$

Where,

Q = flow rate of vehicles (PCU/h)
U = Speed (km/h)
K = Density (PCU/km)

Table 3.7: Sample set of Flow, Speed and Density data

Traffic Flow (PCU/h/l)	Traffic stream Speed (km/h)	Density (PCU/km/l) = Flow/Speed
1707	42	40
1924	45	42
1932	45	43
1967	45	43
1949	45	44
1838	42	44
2192	48	45
1755	38	46
1859	39	48
2259	47	48
2096	43	48
1798	35	51
1939	38	52
2056	40	52

The models reviewed in section 2.6 were fitted to the 5-min speed-density points of the study data set and are shown in Figure 3.6. The models were fitted to the existing speed-density data by minimizing the squared sum of errors (SSE) between the actual speed and the predicted model speed.

$$SSE_{min} = min \sum (v_i - v_i') \tag{26}$$

where v_i' is the predicted speed from the considered model.

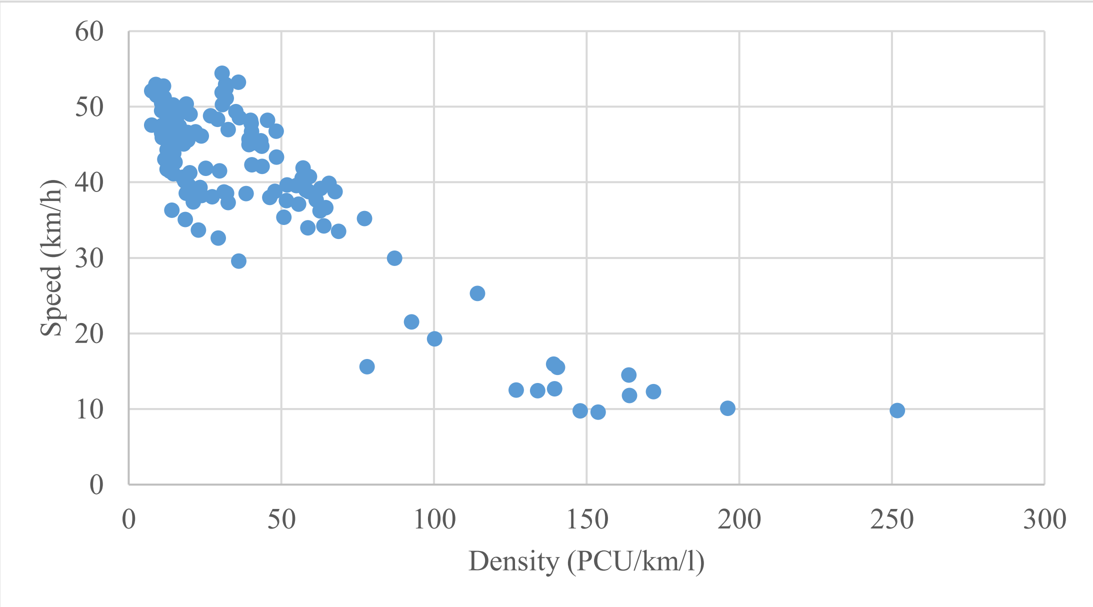


Figure 3.5: Speed-density data plot

Table 3.8 depicts the models fitted to the data, the R-Squared values of the fit and the calibrated models. Drake's model showed the best fit with a R-squared value of 0.81.

Table 3.8: Fitted models to 5-min interval speed-density data

Fitted model	Equation	R-Squared value	Calibrated model equation
Greenshields' model	(2.5)	0.77	$U = 50.23\left(1 - {}^{K}/_{223.95}\right)$
Greenberg's model	(2.6)	0.62	$U = 10.58 \ln\left({}^{1393.22}/_{K}\right)$
Underwood's model	(2.7)	0.76	$U = 52.77e^{(-K/145.08)}$
Pipes-Munjal's model	(2.8)	0.77	$U = 50.73(1 - (K/227.24)^{0.96})$
Drake's model	(2.9)	**0.81**	$\boldsymbol{U = 47.07\, e^{-0.5(K/90.42)^2}}$

3.2.5 Capacity Determination method

Next, based on the fitted model coefficients the speed-flow model was derived. The fitted Drakes model is shown in equation (3.2),

$$U = 47.07\, e^{-0.5\left(\frac{K}{90.42}\right)^2} \tag{3.2}$$

Where U is the speed and K is the density. Combining this with the fundamental flow equation (2.4) the relationship between the flow (Q) and the speed (U) is shown in equation (3.3),

$$Q = 90.42U\left\{2\sqrt{ln(47.07) - ln(U)}\right\} \tag{3.3}$$

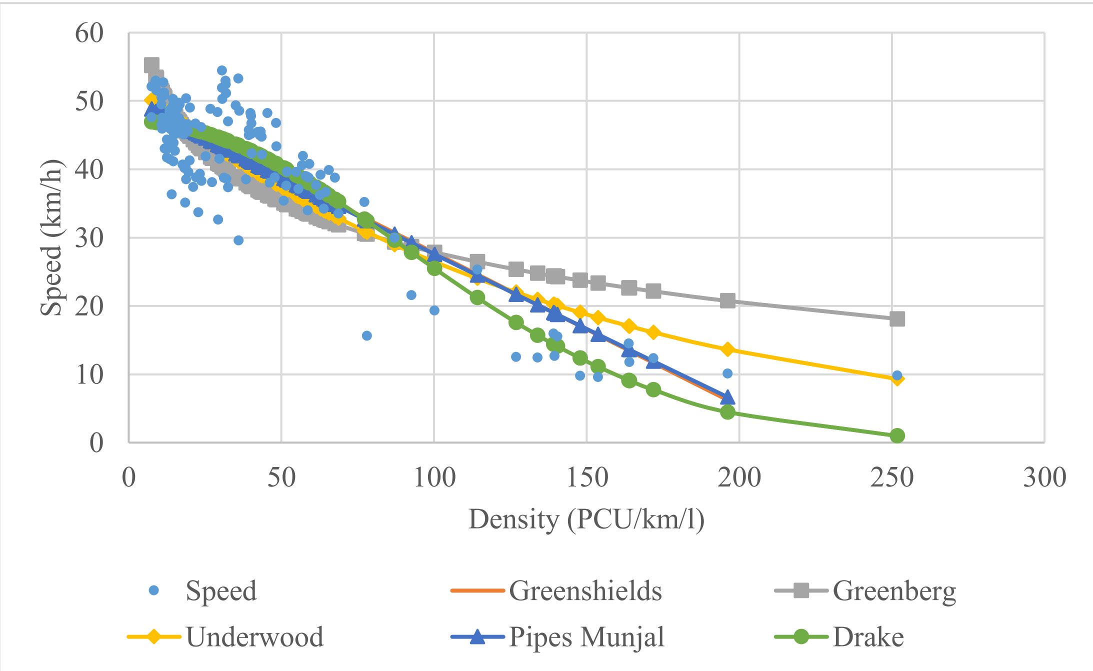


Figure 3.6: 5-min interval speed-density plot with calibrated models

Figure 3.7 shows the speed-flow data plot and equation (3.3) (Drake's fitted speed-flow curve). Based on this curve the capacity of the section is estimated to be 2581 PCU/h/l. The speed at capacity is 29 km/h.

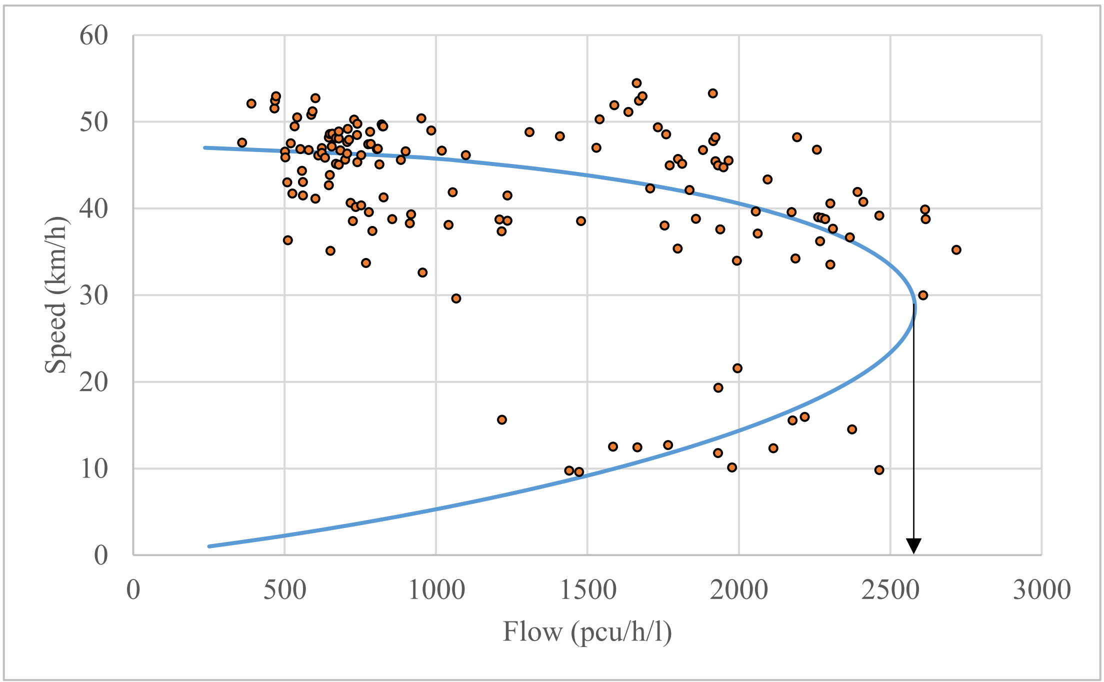


Figure 3.7: Speed-flow data with Drake's flow curve

3.2.6 Determination of Capacity based 15-min flow data

The capacity derived in the previous section is based on 5-min interval data. To evaluate if the time interval has an effect on derived capacity the same procedure is followed for 15-min interval data. The 5-min flow and speed data are aggregated to 15-mins and a similar analysis is carried out.

Similar to the previous analysis traffic stream models were fitted and the best fit model was determined by carrying out SSE minimization technique. The calibrated models and R^2 values for 15-min interval speed-density data are shown in Table 3.9. The best fit model was established as Drake's model with a R-squared value of 0.81. Equation (3.4) depicts the fitted Drake's model to the 15-min speed-density data set.

Table 3.9: Fitted models to 15-min interval speed-density data

Fitted model	Equation	R-Squared value	Calibrated model equation
Greenshields' model	(2.5)	0.78	$U = 53.31\left(1 - {K}/{200.52}\right)$
Greenberg's model	(2.6)	0.61	$U = 10.51\ \ln\left({1445.11}/{K}\right)$
Underwood's model	(2.7)	0.75	$U = 53.49e^{(-K/136.63)}$
Pipes-Munjal's model	(2.8)	0.78	$U = 50.45(1 - (K/194.89)^{1.08})$
Drake's model	(2.9)	**0.81**	$\boldsymbol{U = 47.94\ e^{-0.5(K/81.36)^2}}$

$$U = 47.94e^{-0.5\left(\frac{K}{81.36}\right)^2} \tag{3.4}$$

The fitted Drake's model and the 15-min speed-density data plot is shown in Figure 3.8.

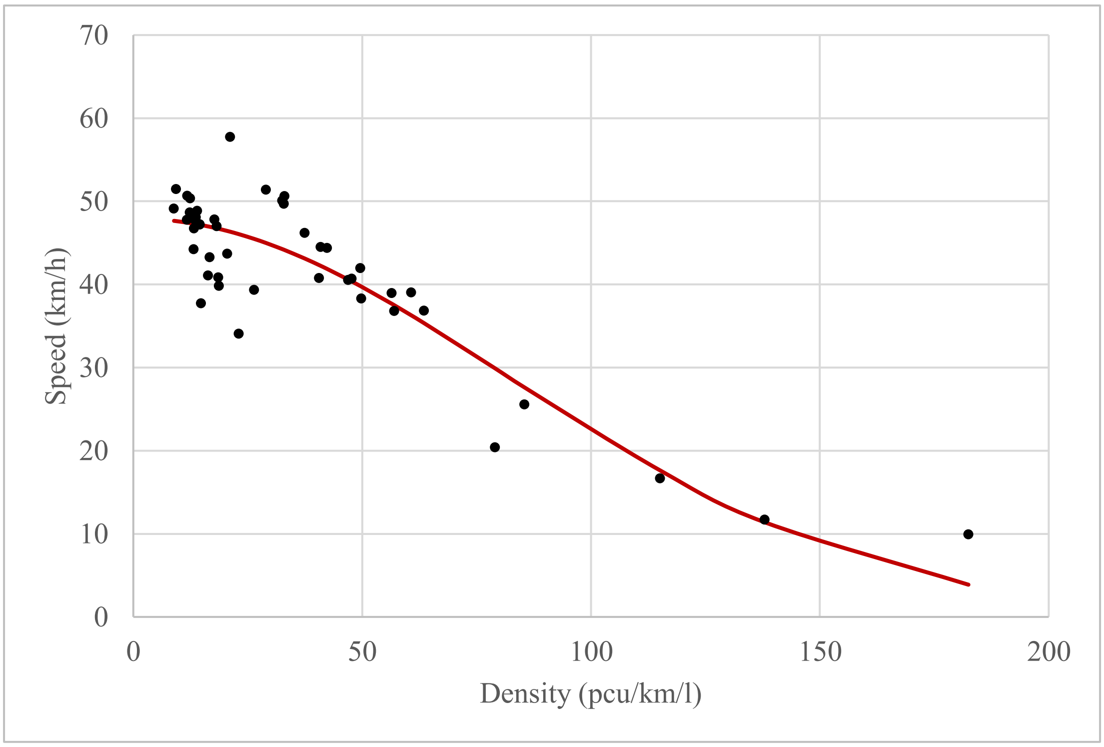


Figure 3.8: 15-min speed-density plot and fitted curve

Similar to the previous section equation (3.4) is combined with equation (2.4) (fundamental flow equation) to obtain the speed-flow model shown in equation (3.5),

$$Q = 81.36U\left\{2\sqrt{[ln(47.94) - ln(U)]}\right\} \quad (3.5)$$

Equation (3.5) is plotted on the speed-flow graph shown in Figure 3.9. Based on the equation the maximum flow i.e. capacity is 2366 PCU/h/l. The speed at capacity 21 km/h. Hence it is observed that the 15-min flow capacity is lesser than the 5-min capacity of 2581 PCU/h/l. The 15-min Capacity value is a more realistic indicator of the capacity as 5-min flowrate is not sustainable over longer periods of time.

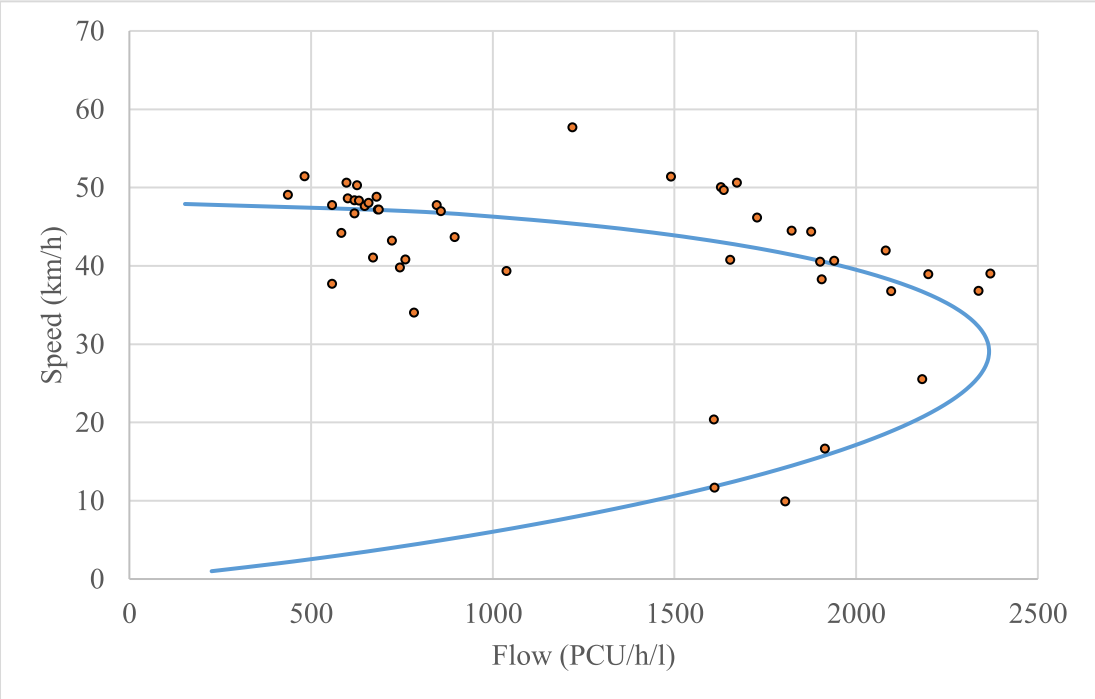


Figure 3.9: Drakes speed-flow curve and 15-min speed-flow data

3.3 Traffic Data collection method – Comparative study

The primary data types required for this study are vehicle flow data, speed data and density data. Given the interrelationship between the three data types as shown in equation (2.4), collection of two data types is sufficient. Since traffic density is not directly acquirable, vehicle flow data and speed data were the two types of data chosen for collection.

Since the research study requires a significant quantity of flow and speed data, the video-based method used in the pilot study (section 3.2) is not best suited for data collection. This is because the time consumed to extract flow and speed data from the video is very high. Hence an alternate method of data collection was explored.

As discussed in the literature review there are different data collection methods available including manual data collection methods, video-based data collection methods, radar-based data collection methods, Google traffic data etc. Hence to determine the best method for data collection a comparative study incorporating available types of data collection methods was carried out.

3.3.1 Evaluated Data collection methods

The TRAZER software (video-based automated data collection method), TIRTL instrument (radar-based data collection method), and Google Distance Matrix (GDM) API (Application Programming Interface) method were examined in this study.

To evaluate the selected methods of automated data collection, traffic surveys were conducted simultaneously using each of the respective methods along with a manual traffic count. An analysis was conducted comparing the manually collected data with the automatically collected data. The manual count was verified by counting vehicles on recorded video clips. A similar method was followed to verify the speed data collected by the automated methods as well.

3.3.2 TRAZER Software traffic data collection

TRAZER is a video processing software developed by KritiKal Solutions, India. The version of the TRAZER software (TRAZER Suite 10) used for this research provides the user with the facility to detect 4 vehicle categories; namely, Light moving vehicles (LMV), Heavy moving vehicles (HMV), Three Wheelers (3W) and Two wheelers (2W). HMV's can be further classified as Buses (BUS) and Trucks (TRUCK) manually. The video to be processed through the software should be recorded in a way such that the camera is placed parallel to the road and aligned to the center of the lane/lanes with the vehicles moving towards the camera as shown Figure 3.10 The recorded video is fed to the software and the software is calibrated by adding the geometric details of the road stretch where the video is recorded. Once the video is processed TRAZER gives classified vehicle flow, speed, and trajectory data. It also offers options to delete, reclassify and add vehicles to its output; thereby giving the user the ability to rectify software errors and raise the accuracy of the count to a 100%. Mallikarjuna et. al (2009) used the TRAZER software to collect classified traffic flows, average vehicle occupancy, and average speeds. They observed that the detection accuracy depended upon the placement of the video camera with respect to the road. If the camera position is not along the center of the road the detection accuracy decreases [73].

The TRAZER software provides three reports in the form of .CSV files. The three reports are,

- Vehicle flow report
- Vehicle occupancy report
- Vehicle trajectory report

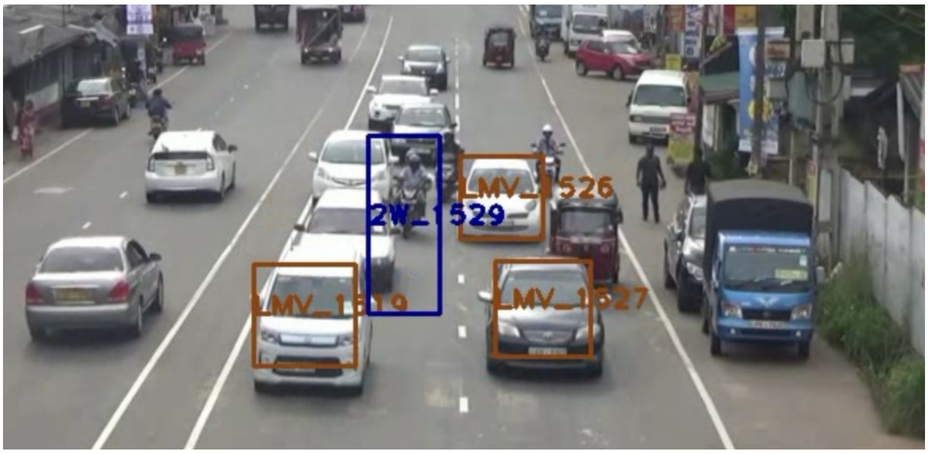


Figure 3.10: TRAZER video detection – A4 highway, Pannipitiya

3.3.3 TIRTL Software traffic data collection

The TIRTL consists of two units, the transmitter and the receiver. Each unit should be placed on either side of the road, close to the edge of the carriageway. IR beams are transmitted across these two units. As the beams get interrupted by the wheels of passing vehicles the instrument identifies the information such as the lane on which the vehicle is travelling, speed, direction, axel width, wheelbase etc. TIRTL classifies vehicles into fifteen categories which are as follows; bicycles, cycle rickshaws, two-wheelers, three-wheelers, tractors, tractors with trailers, SCV (2 axle small commercial vehicles), LMV (2 axle light motor vehicles), LCV (2 axle light commercial vehicles), MCV (2 axle rigid truck or bus), HCV (3 axle rigid truck or bus), HCV (3 axle articulated truck or bus), MAV (rigid truck), MAV (articulated truck) and OSV (oversized truck). TIRTL provides its data in the form of a spreadsheet which can be easily exported to computer software for analysis purposes.

3.3.4 Google Distance Matrix API method

Discussed in Literature review section 2.10.5.

3.3.5 Results of Data collection method Comparative study

The complete methodology and analysis of the comparative study carried out to evaluate the suitability of the TIRTL, TRAZER software and Google Distance Matrix API method to collect traffic data is attached in APPENDIX A.

When comparing the flow estimating capabilities of the TIRTL instrument and the TRAZER software, the TIRTL instrument had an accuracy upwards of 85% and the TRAZER software had an accuracy of 80%. The accuracy of the TRAZER estimation can be brought up to a 100% through manual correction but this proved to be a tedious activity (approximately 4-5 hours of work per hour of traffic data).

When considering the practical usability of the two methods, the TRAZER method once again proved to be challenging. This was due to the fact that the video had to be captured parallel to the road from a high point for the software to accurately capture the vehicle flow (see Figure 3.11). This was not practical considering the roads sections to be surveyed for the research study. The TIRTL instrument on the other hand was comparatively easier to set up as it had to be placed on either side of the road. However, this wasn't possible when a raised center median was present on the road as with most multilane roads in Sri Lanka. Further the accuracy of flow estimation was reduced with the increase in carriageway width.

Considering the speed estimation capabilities of the methods tested, all had acceptable accuracy levels considering the research requirement. When comparing individual speeds, the TRAZER software was a better predictor between the TRAZER software and the TIRTL instrument. When considering the traffic stream speeds the GDM method provided high accuracy (MAPE 1.7%).

Considering the practicality and accuracy of the studied methods the GDM method was selected to collect traffic stream speed data considering its convenience along with manual flow data collection.

Figure 3.11: TRAZER sample video capture point

4 CAPACITY DEVELOPMENT STUDY

4.1 Data Collection Methodology

The data collection method adopted for the study is manual data collection for traffic flow data and GDM for traffic stream speed data. Enumerators were employed to collect the traffic flow data and given data sheets to enter the flow counts (sample data collection sheet attached in APPENDIX B) traffic stream was classified as shown below,

1. Motorcycle
2. Three-Wheeler
3. Car
4. Van
5. Utility Vehicle
6. Light Goods Vehicle (Mini Truck)
7. Medium Goods Vehicle (Medium Truck)
8. Heavy Goods Vehicle (Large Truck)
9. Multi-axle Vehicle
10. Minibus
11. Large Bus

The flow data were collected in 5- and 15-min intervals. The data recorded in the sheets by the enumerators were then entered into an excel file unique to each location surveyed.

Speed data was collected simultaneously using the GDM method as explained in the previous section. The speeds were collected in 1- or 5- min intervals and averaged to fit the 5- or 15- minute flow intervals. The speed data file is created in the format of a .csv file. Figure 4.1 shows the format of a typical output data sheet.

Direction	GPS catchment coordinates				Distance (m)	Travel time (s)	Speed (km/h)	Timezone	Date and time
tocmb	6.68051	79.9218	6.68423	79.92	457	44	37.39	SL/Colomb	9/26/2018 17:08
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:08
tocmb	6.68051	79.9218	6.68423	79.92	457	44	37.39	SL/Colomb	9/26/2018 17:10
fromcmb	6.68423	79.92	6.68051	79.9218	457	43	38.26	SL/Colomb	9/26/2018 17:10
tocmb	6.68051	79.9218	6.68423	79.92	457	45	36.56	SL/Colomb	9/26/2018 17:15
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:15
tocmb	6.68051	79.9218	6.68423	79.92	457	45	36.56	SL/Colomb	9/26/2018 17:20
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:20
tocmb	6.68051	79.9218	6.68423	79.92	457	45	36.56	SL/Colomb	9/26/2018 17:25
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:25
tocmb	6.68051	79.9218	6.68423	79.92	457	45	36.56	SL/Colomb	9/26/2018 17:30
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:30
tocmb	6.68051	79.9218	6.68423	79.92	457	46	35.77	SL/Colomb	9/26/2018 17:35
fromcmb	6.68423	79.92	6.68051	79.9218	457	44	37.39	SL/Colomb	9/26/2018 17:35

Figure 4.1: Output file from GDM speed data collection script

In addition to flow data and speed data, the roadway characteristics of the locations surveyed were also recorded. The collected secondary details are as follows,

1. Median type: The type of median available along the surveyed road section. There are primarily two types of road medians; raised medians and painted centerlines (see Figure 4.2).

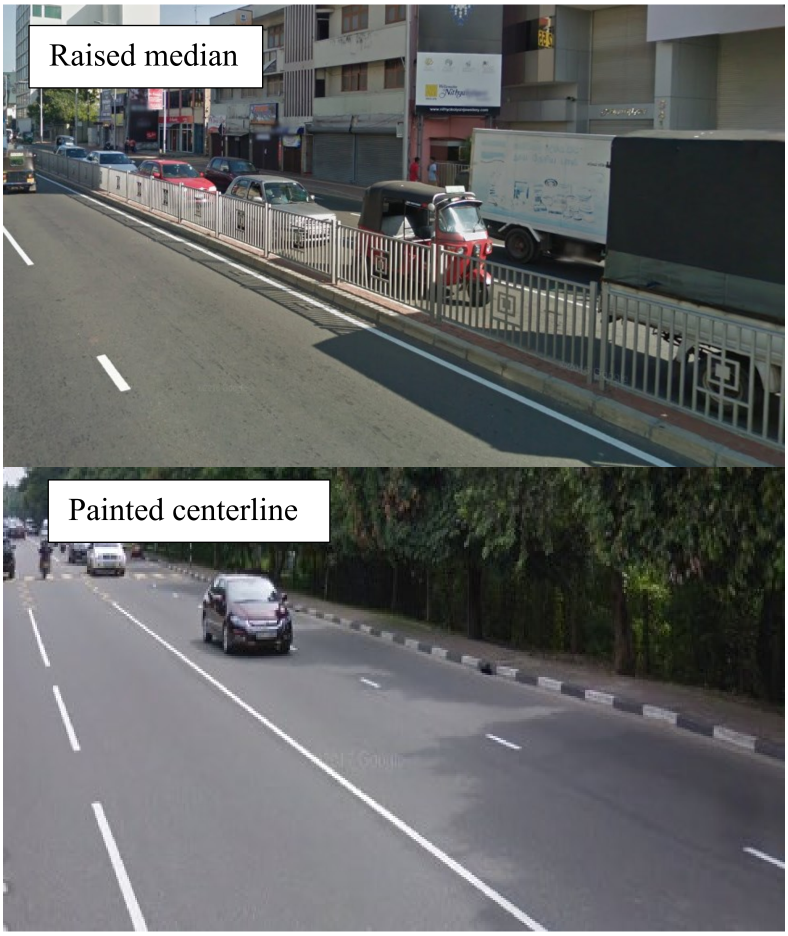


Figure 4.2: Median types on multi-lane roads

2. Number of lanes: This is the number lanes per direction of travel. For multilane roads this will be 2 or more lanes.

3. Lane width: This is the width of an individual lane along the road (see Figure 4.3).

4. Directional width: The total width of all the lanes per direction (see Figure 4.3).

5. Effective directional width: The effective directional width is the width of the road available for vehicles to use. In some road sections part of the road is blocked by parked vehicles. Hence a parameter called effective directional width is introduced (see Figure 4.3).

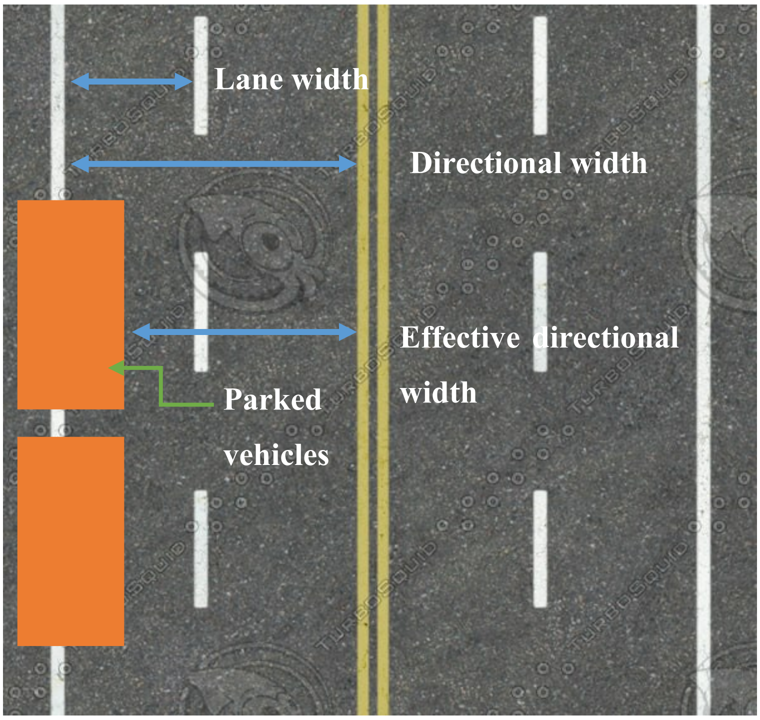


Figure 4.3: Lane width details

6. Effective lane width: This is the width attained by dividing the effective directional width by the number of lanes per direction.

7. Shoulder type: Shoulder is the section on the immediate left to the left most lane of travel. There are three types of shoulders; Hard shoulder, soft shoulder and curb. A shoulder is said to be a hard shoulder when the shoulder is constructed using the same materials as the road pavement (i.e. asphalt concrete). A soft shoulder is when the shoulder is constructed using soil. A curb is a raised edge generally constructed as a pedestrian walkway. (see Figure 4.4)

Figure 4.4: Shoulder types available on multi-lane roads

8. Shoulder width: Width of the shoulder.

9. Lateral Clearance: Distance from pavement edge to first obstruction (boundary wall, light post etc.)

10. Built environment: The built environment is the environment along the side of the road. This is divided into three categories for this study. The three types are Urban, Sub-urban and Rural. This was categorized based on the percentage of built land along the road in a 400m section across the survey point.

- Urban: Built up area > 70%
- Sub-urban: Built up area 20-70%
- Rural: Built up area <20%

11. Access point density: The access point density is taken as number of access roads and center median gaps along a 400m section across the survey point.

4.2 Capacity Study Survey location data

Surveys were carried out across 85 locations in the western province of Sri Lanka. This includes 67 four-lane sections, 15 six-lane sections, and 3 eight-lane sections. Of these, 50 locations were used for model development. A map of a set of locations is shown in Figure 4.5. Details of a sample of a section is shown in Table 4.1. Complete details of surveyed locations are attached in APPENDIX C.

Table 4.1: Details of Loc_1

Location ID	Loc_1
Location coordinates	6.954898, 79.882329
Median type	Median separated
Number of lanes per direction	2
Lane width (m)	3.5
Directional width (m)	7
Effective directional width (m)	7 (no parking/obstructions)
Effective lane width (m)	3.5
Shoulder type	Curb
Shoulder width	-
Lateral clearance (m)	2
Built Environment	Rural
Access point density (accesses per 400m)	0

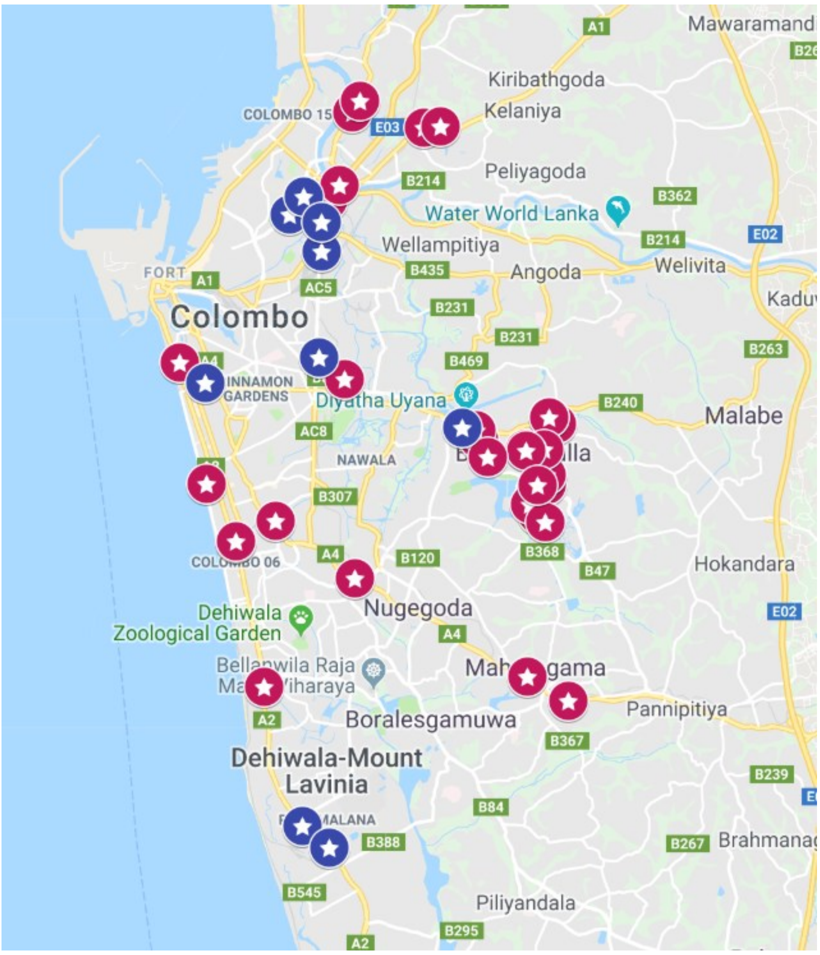
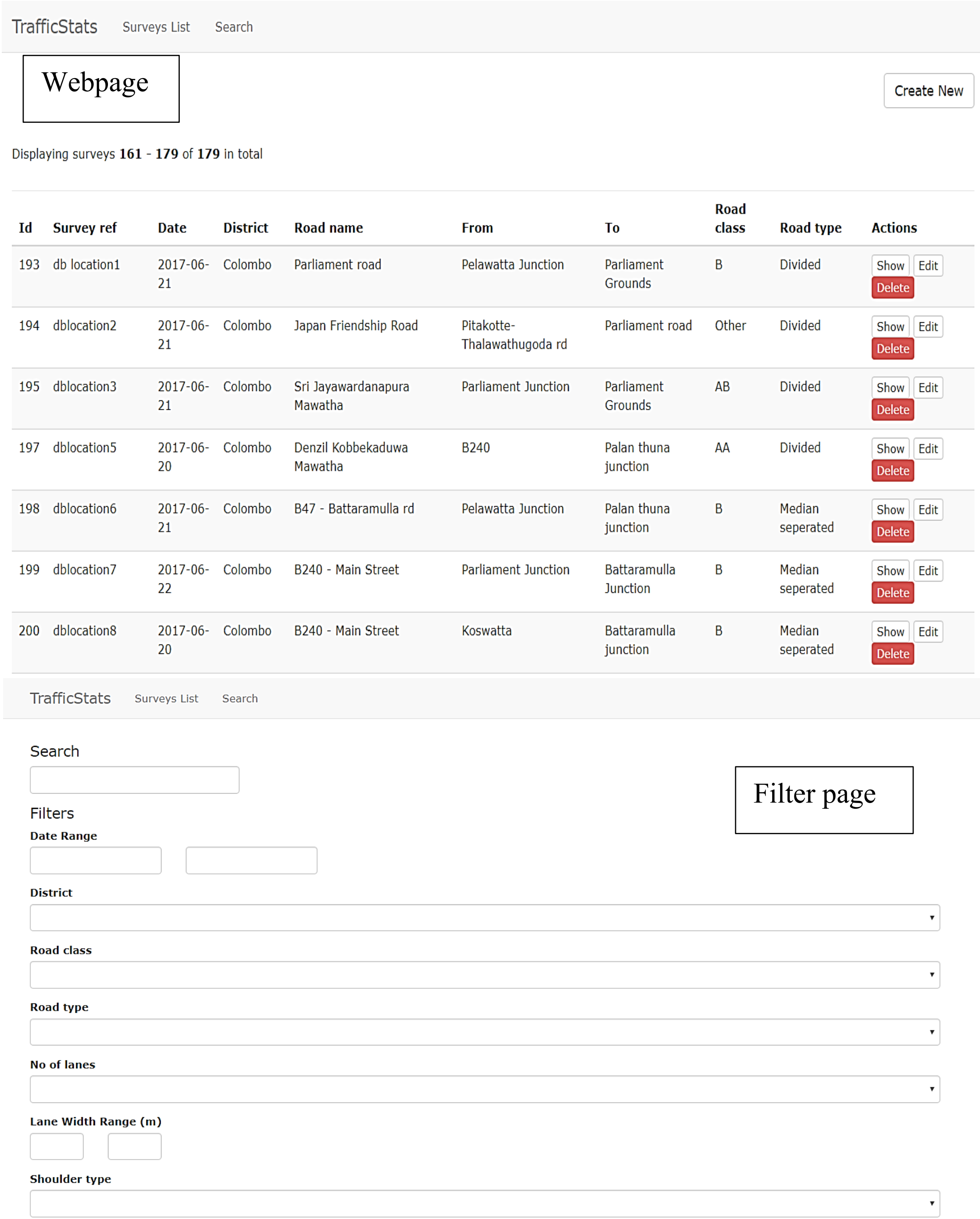


Figure 4.5: Survey locations

4.3 Online Database for Data Storage

A web database was created to store and retrieve location data and flow data of the surveyed locations. This was used as a tool to sort locations by their geometric features. Figure 4.6 shows the homepage (top) and the filter page (bottom) of the web database. The address of the database is: http://188.166.220.159



TrafficStats Surveys List Search

Webpage

Create New

Displaying surveys **161** - **179** of **179** in total

Id	Survey ref	Date	District	Road name	From	To	Road class	Road type	Actions
193	db location1	2017-06-21	Colombo	Parliament road	Pelawatta Junction	Parliament Grounds	B	Divided	Show Edit Delete
194	dblocation2	2017-06-21	Colombo	Japan Friendship Road	Pitakotte-Thalawathugoda rd	Parliament road	Other	Divided	Show Edit Delete
195	dblocation3	2017-06-21	Colombo	Sri Jayawardanapura Mawatha	Parliament Junction	Parliament Grounds	AB	Divided	Show Edit Delete
197	dblocation5	2017-06-20	Colombo	Denzil Kobbekaduwa Mawatha	B240	Palan thuna junction	AA	Divided	Show Edit Delete
198	dblocation6	2017-06-21	Colombo	B47 - Battaramulla rd	Pelawatta Junction	Palan thuna junction	B	Median seperated	Show Edit Delete
199	dblocation7	2017-06-22	Colombo	B240 - Main Street	Parliament Junction	Battaramulla Junction	B	Median seperated	Show Edit Delete
200	dblocation8	2017-06-20	Colombo	B240 - Main Street	Koswatta	Battaramulla junction	B	Median seperated	Show Edit Delete

TrafficStats Surveys List Search

Search

Filter page

Filters

Date Range

District

Road class

Road type

No of lanes

Lane Width Range (m)

Shoulder type

Figure 4.6: TrafficStats web database

4.4 Summary of Location Data

Table 4.2 shows a summary of the locations data were collected for the research.

Table 4.2: Data location highlights

Location code	Road name	Road type	Location Coordinates
Loc_1	A1 - Kandy road	4 lane	6.95489, 79.88232
Loc_2	AB9 - Canada Frienship road	4 lane	7.1638, 79.87917
Loc_3	A3 - Negombo road	4 lane	6.9989, 79.89839
Loc_4	A3 - Negombo road	4 lane	6.9989, 79.89839
Loc_5	Japan Sri Lanka Friendship road	4 lane	6.88329, 79.92646
Loc_6	B240 - Sri Jayawardanepura mawatha	6 lane	6.90322, 79.90876
Loc_7	B240 - Sri Jayawardanepura mawatha	6 lane	6.90322, 79.90876
Loc_8	Denzil Kobbekaduwa mawatha	4 lane	6.89871, 79.92608
Loc_9	AB15 - Sri Jayawaradanepura mawatha	4 lane	6.90086, 79.9121
Loc_10	New Kelani Bridge road	4 lane	6.95077, 79.8751
Loc_11	AB15 - Sri Jayawaradanepura mawatha	4 lane	6.89715, 79.91425
Loc_12	AB15 - Sri Jayawaradanepura mawatha	4 lane	6.89715, 79.91425
Loc_13	Japan Sri Lanka Friendship road	4 lane	6.88329, 79.92646
Loc_14	Japan Sri Lanka Friendship road	4 lane	6.88329, 79.92646
Loc_15	Dharmapala Mawatha	8 lane	6.91267, 79.85329
Loc_16	B533 - New Parliament road	4 lane	6.89091, 79.92488
Loc_17	AB15 - Sri Jayawaradanepura mawatha	4 lane	6.90086, 79.9121
Loc_18	B533 - New Parliament road	4 lane	6.891, 79.92689
Loc_19	B533 - New Parliament road	4 lane	6.89091, 79.92488
Loc_20	AC5 - Baseline road	6 lane	6.91821, 79.87775
Loc_21	Sirimavo Bandaranayake Mawatha	6 lane	6.9489, 79.87174
Loc_22	A4 - High Level road	4 lane	6.85023, 79.92265
Loc_23	A4 - High Level road	4 lane	6.8448, 79.93135
Loc_24	A4 - High Level road	4 lane	6.8448, 79.93135
Loc_25	B47 - Battaramulla-Pannipitiya road	4 lane	6.89853, 79.92228
Loc_26	B240 - Sri Jayawardanepura mawatha	4 lane	6.90304, 79.91175
Loc_27	B47 - Battaramulla-Pannipitiya road	4 lane	6.89347, 79.92703
Loc_28	Japan Sri Lanka Friendship road	4 lane	6.88329, 79.92646
Loc_29	A2- Galle road	4 lane	6.6652, 79.92981
Loc_30	A2- Galle road	4 lane	6.6652, 79.92981
Loc_31	A2- Galle road	4 lane	6.8786, 79.85999
Loc_32	Marine Drive	4 lane	6.89126, 79.85376
Loc_33	Marine Drive	4 lane	6.89126, 79.85376
Loc_34	B533 - New Parliament road	4 lane	6.891, 79.92689
Loc_35	A1 - Kandy road	4 lane	6.95186, 79.87973
Loc_36	A1 - Kandy road	4 lane	6.95186, 79.87973
Loc_37	Denzil Kobbekaduwa mawatha	4 lane	6.90443, 79.92883
Loc_38	Denzil Kobbekaduwa mawatha	4 lane	6.90443, 79.92883

Table 4.2: Data location highlights (Continued)

Loc_39	B62 - Cotta Road	4 lane	6.91368, 79.88335
Loc_40	New Kelani Bridge road	4 lane	6.95077, 79.8751
Loc_41	A3 - Negombo road	4 lane	6.97298, 79.88671
Loc_42	A3 - Negombo road	4 lane	6.97298, 79.88671
Loc_43	A3 - Negombo road	4 lane	6.97069, 79.88489
Loc_44	A4 - High Level road	4 lane	6.85023, 79.92265
Loc_45	B240 - Kaduwela road	4 lane	6.90561, 79.92718
Loc_46	Denzil Kobbekaduwa mawatha	4 lane	6.89871, 79.92608
Loc_47	B240 - Sri Jayawardanepura mawatha	4 lane	6.90304, 79.91175
Loc_48	B240 - Kaduwela road	4 lane	6.90561, 79.92718
Loc_49	A3 - Negombo road	4 lane	6.97069, 79.88489
Loc_50	A1 - Kandy road	4 lane	6.96775, 79.90388
Loc_51	A1 - Kandy road	4 lane	6.96775, 79.90388
Loc_52	A1 - Kandy road	6 lane	6.96718, 79.90061
Loc_53	A1 - Kandy road	6 lane	6.96718, 79.90061
Loc_54	B62 - Cotta Road	4 lane	6.91368, 79.88335
Loc_55	B240 - Kaduwela road	4 lane	6.90561, 79.92718
Loc_56	B240 - Kaduwela road	4 lane	6.90561, 79.92718
Loc_57	A4 - Havelock road	4 lane	6.88352, 79.86841
Loc_58	B47 - Battaramulla-Pannipitiya road	4 lane	6.89347, 79.92703
Loc_59	B47 - Battaramulla-Pannipitiya road	4 lane	6.89347, 79.92703
Loc_60	A4 - Havelock road	4 lane	6.88352, 79.86841
Loc_61	A2- Galle road	4 lane	6.8786, 79.85999
Loc_62	B47 - Battaramulla-Pannipitiya road	4 lane	6.89853, 79.92228
Loc_63	B47 - Battaramulla-Pannipitiya road	4 lane	6.89853, 79.92228
Loc_64	B47 - Battaramulla-Pannipitiya road	4 lane	6.89347, 79.92703
Loc_65	A4 - High Level road	4 lane	6.871300, 79.88554
Loc_66	A4 - High Level road	4 lane	6.871300, 79.88554
Loc_67	B47 - Battaramulla-Pannipitiya road	4 lane	6.89853, 79.92228
Loc_68	A2- Galle road	4 lane	6.91705, 79.84777
Loc_69	A2- Galle road	4 lane	6.91705, 79.84777
Loc_70	AC5 - Baseline road	6 lane	6.91821, 79.87775
Loc_71	B240 - Sri Jayawardanepura mawatha	6 lane	6.94123, 79.87835
Loc_72	Sirimavo Bandaranayake Mawatha	6 lane	6.9489, 79.87174
Loc_73	A2- Galle road	6 lane	6.818353, 79.87435
Loc_74	A2- Galle road	4 lane	6.8786, 79.85999
Loc_75	A2- Galle road	4 lane	6.84791, 79.866
Loc_76	A2- Galle road	4 lane	6.84791, 79.866
Loc_77	B240 - Sri Jayawardanepura mawatha	6 lane	6.94123, 79.87835
Loc_78	A2- Galle road	6 lane	6.813563, 79.88009
Loc_79	A2- Galle road	6 lane	6.813563, 79.88009
Loc_80	AB11 - New Galle road	4 lane	6.76838, 79.88315
Loc_81	Sirimavo Bandaranayake Mawatha	6 lane	6.95265, 79.87463
Loc_82	Sirimavo Bandaranayake Mawatha	6 lane	6.95265, 79.87463

Table 4.2: Data location highlights (Continued)

Loc_83	A1 - Baseline road	8 lane	6.94734, 79.87835
Loc_84	A1 - Baseline road	8 lane	6.94734, 79.87835
Loc_85	AB11 - New Galle road	4 lane	6.76838, 79.88315

4.5 Capacity Estimation Methodology

The capacity estimation methodology utilized was similar to what was explained in section 3.2. A breakdown of the framework is described below,

1 Collection of 15-min classified flow data and speed data
2 Conversion of classified flows to homogeneous flows using PCU factors
3 Developing density data using fundamental traffic flow equation (Q=UK), speed and flow data
4 Fitting traffic stream model based on speed-density data
5 Computing speed-flow model based on fitted traffic stream model and fundamental traffic flow equation
6 Estimating capacity based on developed speed-flow model

15-min classified flow counts were done by employing enumerators to count the traffic flow. The traffic stream speeds were measured using the GDM method. When measuring stream speeds, the method explained in section 0 was followed. The trap length for a section taken was approximately 200m. 1-min/5-min speed data were aggregated to form 15-min average speed values for each corresponding flow interval.

Average PCU factors developed in the preliminary study (Table 3.5) were used to convert the heterogeneous traffic flows into uniform flows. Greenshields' model was used as the traffic stream model to develop the speed-flow curve as it provided the best overall fit considering all the sections studied.

4.5.1 Capacity Estimation – Example

Location_41

Step 01: Collection of 15-min classified flow data and speed data

A sample 25 data points from the location is shown in Table 4.3. Column 01 shows the flowrate (aggregate 15- min flow multiplied by 4 to convert to hourly flow/flowrate) and column 04 shows the respective traffic stream speed of each time interval.

Table 4.3: Sample set of data from Location_41

Flowrate (veh/h/l)	Flowrate (PCU/h/l)	Density (PCU/km)	Speed (km/h)
1612	1596	57	28
1690	1625	76	21
1824	1749	73	24
1576	1324	51	26
1976	1506	66	23
2044	1459	76	19
2212	1759	115	15
1904	1533	105	15
1944	1673	96	17
1746	1640	66	25
1242	1481	52	29
1174	1338	43	31
1328	1473	46	32
1266	1486	46	32
1192	1511	48	32
1096	1306	40	33
880	1246	40	32
870	1075	33	32
998	1124	35	32
1036	1282	41	31
942	1198	38	31
1004	1290	41	31
772	914	27	34
714	860	24	35
950	1423	43	33
928	1394	43	32

Step 02: Conversion of classified flows to homogeneous flows using PCU factors

The collected heterogeneous flow was converted to a uniform flow using PCU factors shown in Table 3.5 and the equation (4.1) shown below.

$$Flowrate \ = \ \Sigma \ Count \ of \ i^{th} \ vehicle \ category \ x \ PCU_i \ x \ 4 \qquad (4.1)$$

Table 4.4 shows an example of how a 15-min classified flow is converted to a flowrate using the above mentioned PCU factors and equation (4.1). The final PCU flowrates of the sample set in Table 4.3 is shown in column 02.

Table 4.4: Example conversion of 15-min classified flow to uniform flowrate

Vehicle category	Car	Van	MC	TW	Utility V	Mini Truck	Medium Truck	Large Truck	Multi Axle	Minibus	Large Bus	Total
15min Flow	110	22	127	90	17	3	10	2	1	5	20	407
PCU factor	1	1.2	0.2	0.6	1.7	1	2.5	3.7	6.5	2.3	5.4	
PCU flow	110	26.4	25.4	54	28.9	3	25	7.4	6.5	11.5	108	406.1
PCU flow rate	= 406.1 x 4 (PCU/h/l)											1624

Step 03: Developing density data using fundamental traffic flow equation (Q=UK), speed and flow data

Column 03 of Table 4.3 depicts the respective density values of each 15-min time interval. The density values are calculated using the fundamental traffic flow equation.

Step 04: Fitting traffic stream model based on speed-density data

Greenshields' model was fitted to the existing speed-density data by minimizing the squared sum of errors (SSE) between the actual speed and the predicted model speed as shown in equation (4.2).

$$SSE_{min} = Min \sum (v_i - v_i') \qquad (4.2)$$

where v_i' is the predicted speed from the model.

The fitted Greenshields' model (equation (4.3)) is shown in Figure 4.7. The model showed a good fit with a r-squared value of 0.92.

$$v = 43.1\left(1 - \frac{k}{160.6}\right) \tag{4.3}$$

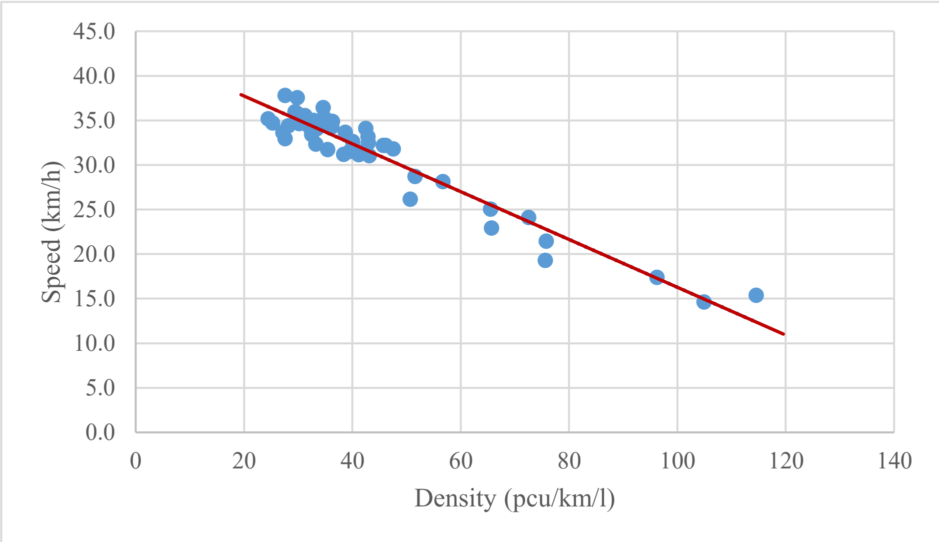


Figure 4.7: Fitted Greenshields' model to speed-density data

<u>Step 05:</u> Computing speed-flow model based on fitted traffic stream model and fundamental traffic flow equation

Using the fitted Greenshields' model (equation (4.3)) and the fundamental traffic flow equation (equation 04) the relationship between the flow and speed for Location_41 was derived. The developed relationship is shown in equation (4.4).

$$Q = 3.7\,(43.1 - v) * v \tag{4.4}$$

<u>Step 06:</u> Estimating capacity based on developed speed-flow model

Based on equation (4.4) the capacity (Q_{max}) for location_41 was 1730 pcu/h/l. The capacity curve is shown graphically in Figure 4.8.

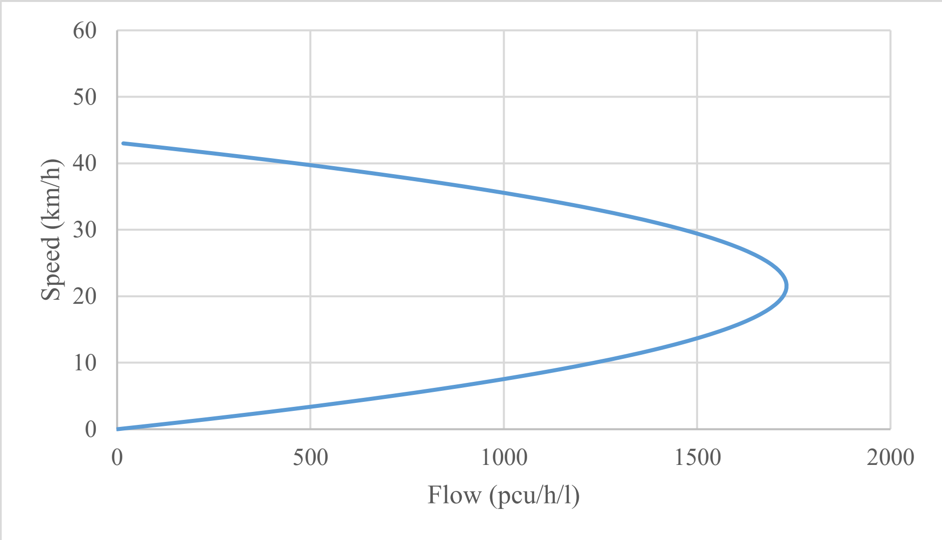


Figure 4.8: Developed speed-flow relationship for location_41

4.5.2 Developed Capacity values

Following the framework explained in section 3.2 the capacity values of 51 sections were derived. The derived capacity values are shown in Table 4.5. The capacity values varied from 2349 pcu/h/l to 1231 pcu/h/l.

Table 4.5: Developed capacity values

Location code	Capacity (pcu/h/l)	Location code	Capacity (pcu/h/l)	Location code	Capacity (pcu/h/l)
Loc_56	2349	Loc_15	1941	Loc_60	1727
Loc_1	2309	Loc_53	1933	Loc_61	1717
Loc_2	2276	Loc_63	1916	Loc_50	1712
Loc_17	2243	Loc_62	1908	Loc_27	1711
Loc_85	2200	Loc_33	1889	Loc_4	1690
Loc_82	2162	Loc_3	1884	Loc_38	1678
Loc_47	2140	Loc_57	1873	Loc_51	1634
Loc_80	2107	Loc_77	1847	Loc_81	1596
Loc_18	2100	Loc_66	1788	Loc_21	1575
Loc_64	2062	Loc_75	1785	Loc_65	1513
Loc_83	2028	Loc_71	1775	Loc_54	1509
Loc_72	1995	Loc_8	1770	Loc_28	1482
Loc_73	1992	Loc_84	1770	Loc_70	1466
Loc_32	1983	Loc_52	1755	Loc_78	1411
Loc_16	1965	Loc_76	1748	Loc_11	1386
Loc_55	1961	Loc_7	1746	Loc_34	1369
Loc_74	1943	Loc_59	1730	Loc_40	1231

5 CAPACITY DATA ANALYSIS

5.1 Capacity Details

Maximum observed lane capacity: 2349 pcu/h/l

Minimum observed lane capacity: 1231 pcu/h/l

Mean capacity: 1829 pcu/h/l

Figure 5.1 depicts the histogram of the derived capacity values.

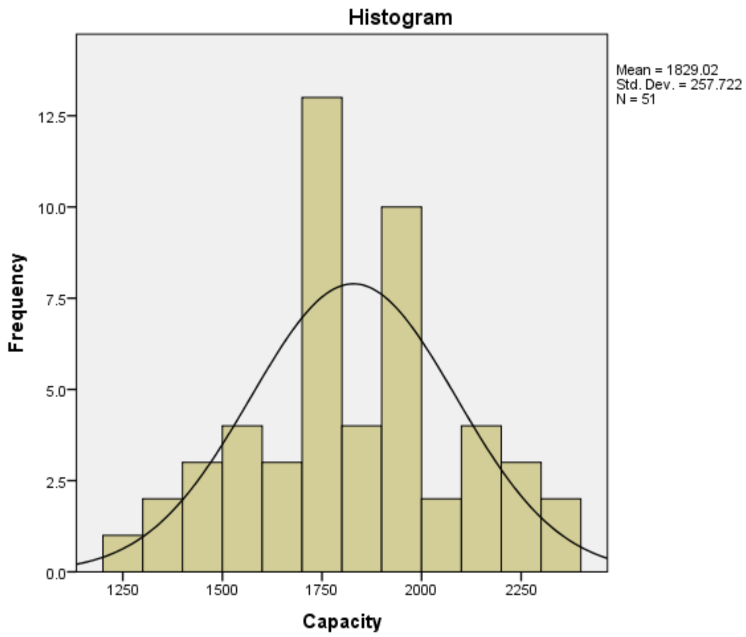


Figure 5.1: Capacity histogram

It was observed that the derived capacities are comparable with capacity values found in literature. The HCM 2010, AustRoads manual, RDA guideline have base capacity values ranging between 1900-2200 PCU/h/l. Hence the mean capacity 1829 PCU/h/l derived in this study is comparable with these values.

5.1.1 Capacity variation with Effective Carriageway width

The variation of capacity of heterogeneous traffic flows with lateral road width or carriageway width is well documented in literature. Hence this was tested with the capacity data set of this study. The effective carriageway width range: 4.2m – 10.2m. Figure 5.2 depicts the plot between the effective carriageway width and directional capacity. It is observed that with the increase in carriageway width the capacity has increased. Using MS Excel software, a linear regression line was drawn, and the coefficient of determination was 0.61 suggesting a strong relationship between the two quantities.

Similarly, lane capacities were plotted against the effective lane widths. This is shown in Figure 5.3. The capacities were segregated by the built environment; Urban, Sub-Urban and Rural. Here it is observed that the increase in capacity with the increase in lane width is more pronounced in Rural roads. The increase in capacity per 1m increase in lane width is approximately 200 PCU/h/l greater in Rural roads than in Sub-Urban and Urban roads.

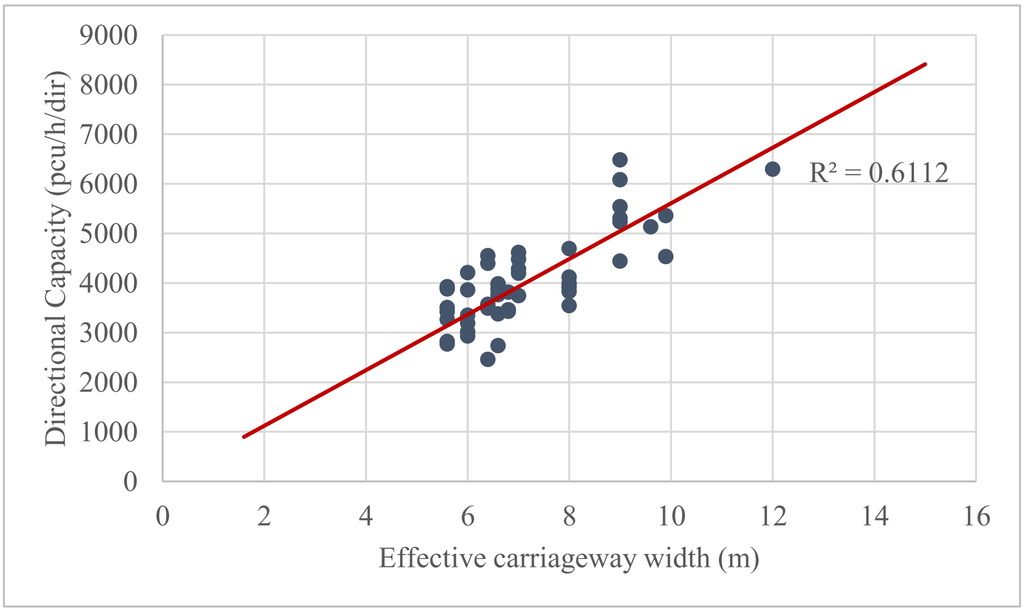


Figure 5.2: Effective carriageway width vs Directional Capacity

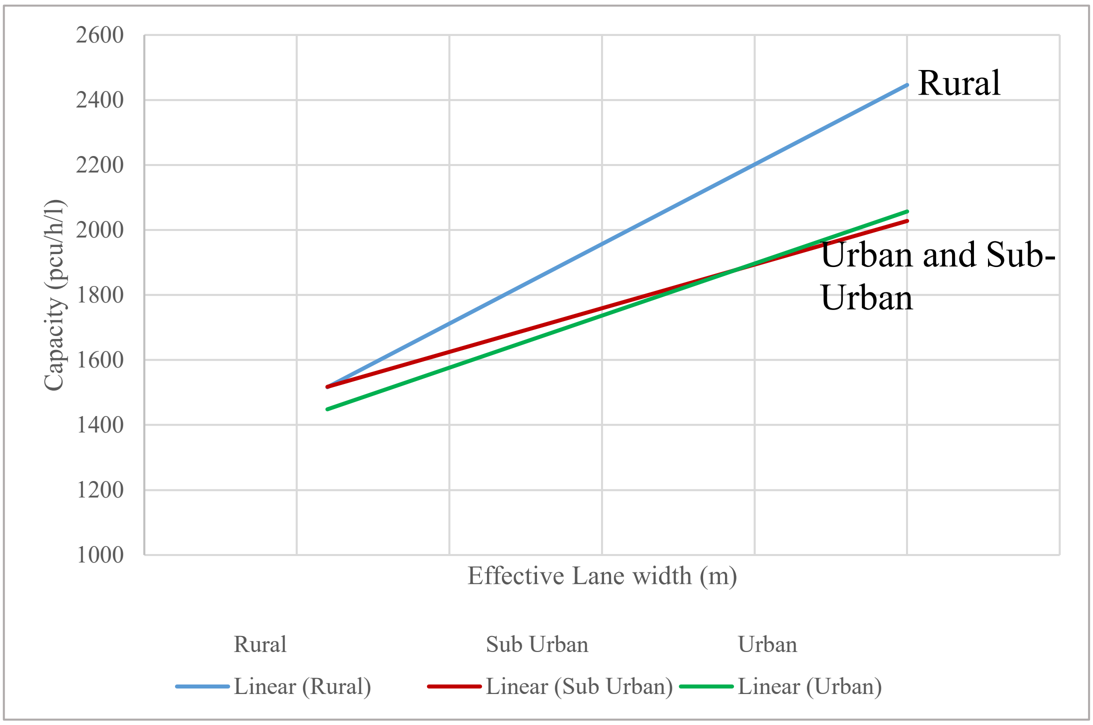


Figure 5.3: Effective Lane width vs Lane capacity

5.1.2 Capacity variation with Built Environment

Figure 5.4 shows the variation of lane capacity in different roadside contexts. As expected, the Rural roads had the highest average capacity and Urban roads had the lowest capacity.

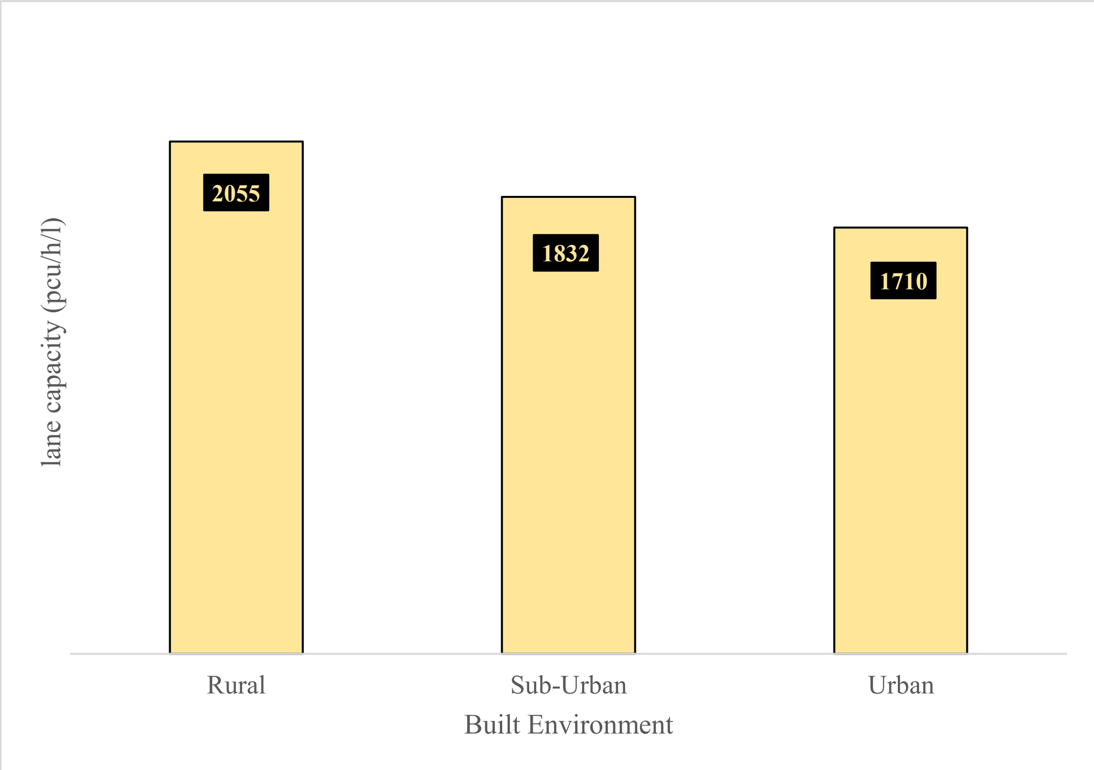


Figure 5.4: Average lane capacity vs Built environment

5.1.3 Capacity variation with Access Point Density

Similar to the built environment the variation in average capacity was tested in terms of the Access point density. The access point density (access points per 400m section) was divided into three ranges as shown below.

Low : <4

Medium : 4-8

High : >8

Figure 5.5 shows the variation in average lane capacity with the change in access point density. As expected with the increase in Access points the lane capacity has reduced. This could be due to the conflicting vehicle movements and the psychological effect it has on drivers compelling them to lower driving speeds.

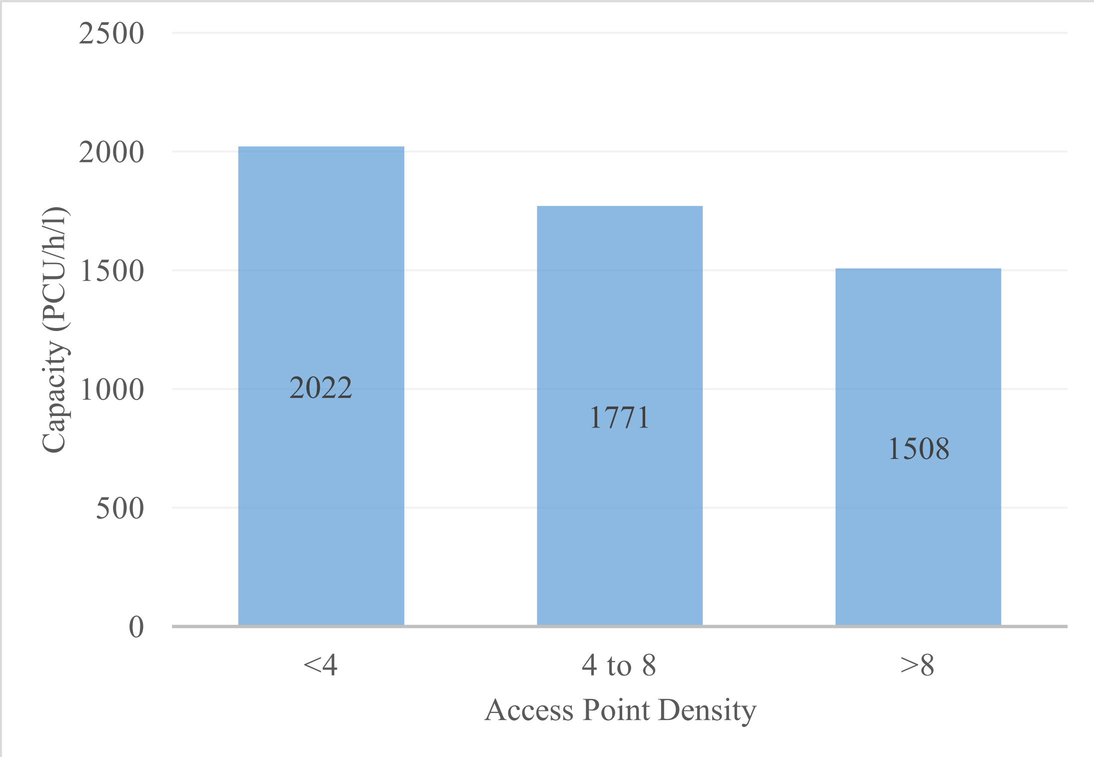


Figure 5.5: Average lane capacity vs Access Point Density

5.2 Capacity model Development methodology

As observed in previous section multilane highway capacity is dependent upon a number of roadway factors. Therefore, an attempt was made to develop a model incorporating these factors. 'IBM SPSS Statistics 24' statistical software was used for this analysis.

5.2.1 Capacity model for 4-lane highways

41 four-lane road sections were analyzed for this study. The independent variables selected based on statistical significance were,

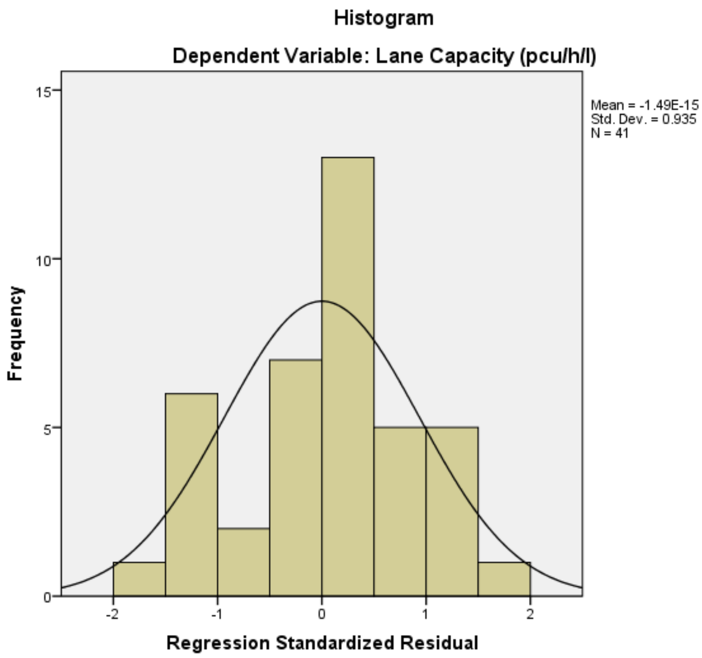
- Effective lane width (2.1m – 4.0m)
- Access point density (0-13 per 400m)
- Median type (Median separated, Divided)

- Built environment (Rural, Sub-Urban and Urban)

Multiple linear regression analysis was performed to develop the capacity model. The pre-requisites to carry out the regression are documented below.

Linearity of independent variables and homoscedasticity of data were checked to satisfaction by visual observation of partial regression plots and the scatterplot between 'Studentized Residuals' and 'Unstandardized Predicted values'. Next the data was checked for multicollinearity (To check if two or more independent variables are highly correlated with each other). This was done by the inspection of correlation coefficients and Tolerance/VIF values. Since there were no correlations greater than 0.7 (maximum value being 0.615) and no Tolerance values lesser than 0.1 [74] (minimum being 0.333) it was established that there was no multicollinearity in the data set. Next the data set was checked for outliers by running the 'Case wise diagnostics' function in the SPSS software. This produced no outliers in the data set (Case's where the standardized residual is greater than ±3 standard deviations). Next the data was checked for leverage points and influential points, both of which were absent in the data set.

The normality of the data was verified by observing the Histogram and the P-P plot of the standardized residuals (see Figure 5.6). Since the mean is close to zero ($= -1.5*10^{-15}$) and Standard deviation is approximately one (= 0.94) the data can be approximated to be normal.



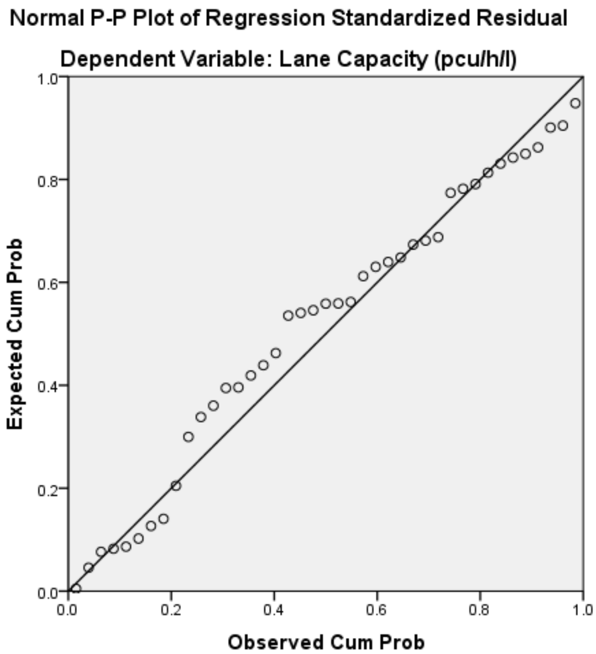


Figure 5.6: Histogram (top) and P-P plot (bottom) of data set

Since the data set is conducive to perform the regression, the analysis was carried out using IBM SPSS software with Effective lane width, Access point density and Built environment as independent variables and the Lane capacity as the dependent variable. The effective lane width and access point density were entered as 'scale' variables and the built environment and median type which are categorical variables were entered as a 'Nominal' variable to the software. The built environment categorical variable which has three levels (Urban, Sub-Urban and Rural) was dummy coded to two dichotomous variables. For the regression analysis the 'Rural' category was taken as the reference

Table 5.1: Model Summary

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.898[a]	.806	.778	117.030	2.344

a. Predictors: (Constant), Median Type, Effective lane width, Sub_Urban, Access point density, Urban

b. Dependent Variable: Lane Capacity (pcu/h/l)

category.

The regression model coefficient of determination (R^2) is 0.81 shown in Table 5.1 indicates that the independent variables account for a major portion of the variance of the dependent variable. Further, it is observed that the independent variables statistically significantly predict Lane capacity from Table 5.2 as the P value is less than 0.05.

Table 5.2: ANOVA table for regression

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1991917.967	5	398383.593	29.088	.000[b]
	Residual	479360.911	35	13696.026		
	Total	2471278.878	40			

a. Dependent Variable: Lane Capacity (pcu/h/l)

b. Predictors: (Constant), Median Type, Effective lane width, Sub_Urban, Access point density, Urban

Table 5.3 indicates the results of the regression analysis. Unstandardized coefficients of the independent/predictor variables and the significance of each of them are highlighted in the table. The significance (p value) of each predictor variable tests the null hypothesis that the coefficient is equal to zero. Hence, since the p-values of each of the predictor variables are less than 0.05, the null hypothesis is rejected, and the variables are accepted to the model.

Table 5.3: Coefficient table of regression

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.
1	(Constant)	1467.120	145.218		10.103	.000
	Effective lane width	189.547	41.111	.392	4.611	.000
	Access point density	-38.859	7.359	-.519	-5.281	.000
	Sub_Urban	-145.775	63.371	-.297	-2.300	.028
	Urban	-206.158	65.044	-.398	-3.170	.003
	Median Type	118.014	57.930	.157	2.037	.049

a. Dependent Variable: Lane Capacity (pcu/h/l)

Therefore the 4-lane capacity model may be written in the form shown in equation (5.1).

$$C_4 = 1467 + 190\ C_L' + 118\ C_M' - 39\ C_A' - \ C_{BE}' \qquad (5.1)$$

Where,

C_4 = 4-Lane Capacity (pcu/h/l)

C_L' = Effective lane width (m)

C_M' = Median Type (0,1)

C_A' = Access point density (per 400m section)

C_{BE}' = Built environment (refer Table 5.4.)

In the developed model the 'Effective lane width (C_L')' is the width of road space available for vehicles to travel in meters and the 'Access point density (C_A')' is the value representing the number of access points within the selected 400m section. For the 'Median Type (C_M')' variable, values 0 or 1 should be substituted for Divided (median-less) and Median separated sections respectively. The values to be entered to the model for the 'Built environment (C_{BE}')' is shown in Table 5.4.

Table 5.4: Built Environment type factors for equation (5.1)

Built Environment type	Value to enter equation (5.1)
Urban	206
Sub-Urban	146
Rural	0

The 4-lane model can be further simplified by combining the C'_{BE} factor to represent values between 0 and 1. This can be done by assigning a coefficient of value 206 to C'_{BE} and allocating the values 0, 0.7 (=146/206), and 1 to indicate rural, sub-urban and urban sections respectively. Hence the updated 4-lane capacity model can be written as shown in equation (5.2).

$$C_4 = 1467 + 190\, C'_L + 118\, C'_M - 39\, C'_A - \ 206\, C'_{BE} \qquad (5.2)$$

Where,

C_4 = 4-Lane Capacity (pcu/h/l)

C'_L = Effective lane width (m)

C'_M = Median Type (0,1)

C'_A = Access point density (per 400m section)

C'_{BE} = Built environment (0, 0.7, 1)

For the 'Median Type (C'_M)' variable, values 0 and 1 should be substituted for Divided (median-less) and Median separated sections respectively.

Figure 5.7 illustrates the variation of lane capacity estimated by the 4-lane capacity model with effective lane width. It is observed that with increase in lane width the lane capacity increases by an amount of 190 pcu/h/l/m. In comparison the HCM lane capacity changes at a rate of 167 pcu/h/l/m and the IHCM lane capacity changes at a rate of 264 pcu/h/l/m. The effect the roadside built environment has on the capacity is also captured in the model. Sub-Urban road sections have a base capacity 146 pcu/h/l lower than Rural road sections. Further, Urban roads have a capacity 206 pcu/h/l lesser than Rural roads. Another factor that negatively impacts capacity is the Access Point

Density of the road section will drop by 39 pcu/h/l. In comparison the HCM guideline states that the capacity will reduce by 13 pcu/h/l per access point per 400m catchment. The median separation is the other factor that alters the model capacity. Road sections with no median separation will have a capacity 118 pcu/h/l lower than those that do.

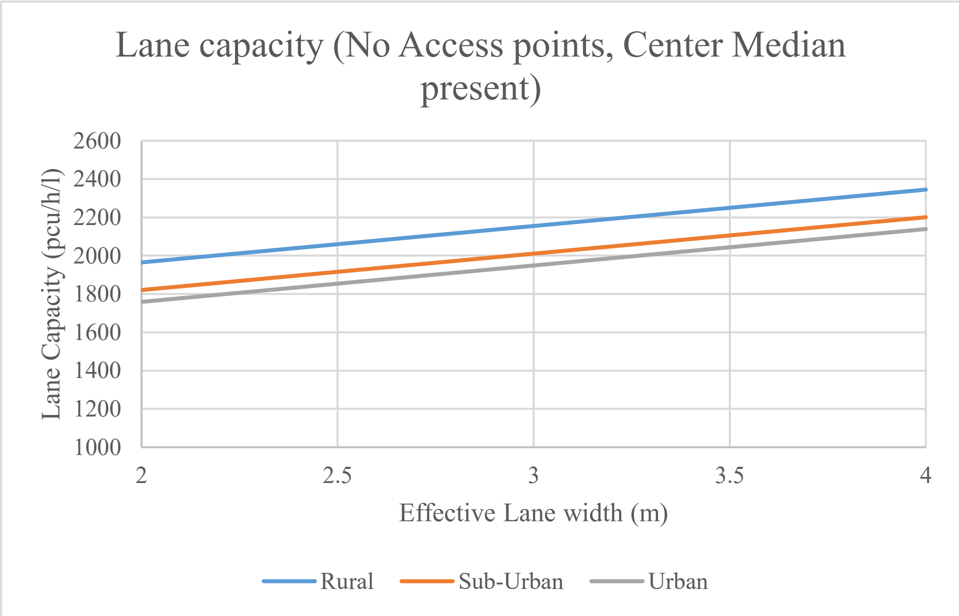


Figure 5.7: 4-lane model Capacity variation with effective lane width

A summary of the 4-lane capacity model is presented in Table 5.5. Interpolation between values is allowed.

Table 5.5: 4-lane model capacity table

Access Point Density	Effective Lane width (m)	Lane Capacity (pcu/h/l)					
		No median Separation			Median Separated		
		Rural	Sub-Urban	Urban	Rural	Sub-Urban	Urban
0	2.0	1847	1703	1641	1965	1821	1759
	2.5	1942	1798	1736	2060	1916	1854
	3.0	2037	1893	1831	2155	2011	1949
	3.5	2132	1988	1926	2250	2106	2044
	4.0	2227	2083	2021	2345	2201	2139
2	2.0	1769	1625	1563	1887	1743	1681
	2.5	1864	1720	1658	1982	1838	1776
	3.0	1959	1815	1753	2077	1933	1871
	3.5	2054	1910	1848	2172	2028	1966
	4.0	2149	2005	1943	2267	2123	2061
4	2.0	1691	1547	1485	1809	1665	1603
	2.5	1786	1642	1580	1904	1760	1698
	3.0	1881	1737	1675	1999	1855	1793
	3.5	1976	1832	1770	2094	1950	1888
	4.0	2071	1927	1865	2189	2045	1983
6	2.0	1613	1469	1407	1731	1587	1525
	2.5	1708	1564	1502	1826	1682	1620
	3.0	1803	1659	1597	1921	1777	1715
	3.5	1898	1754	1692	2016	1872	1810
	4.0	1993	1849	1787	2111	1967	1905
8	2.0	1535	1391	1329	1653	1509	1447
	2.5	1630	1486	1424	1748	1604	1542
	3.0	1725	1581	1519	1843	1699	1637
	3.5	1820	1676	1614	1938	1794	1732
	4.0	1915	1771	1709	2033	1889	1827
10	2.0	1457	1313	1251	1575	1431	1369
	2.5	1552	1408	1346	1670	1526	1464
	3.0	1647	1503	1441	1765	1621	1559
	3.5	1742	1598	1536	1860	1716	1654
	4.0	1837	1693	1631	1955	1811	1749

5.2.2 Capacity model for 6-lane highways

Nine 6-lane sections were available for analysis. The study sections were all median separated and predominantly 'Sub-Urban' hence the Median type and 'Built Environment' variables were not factored into the model.

Similar to the 4-lane capacity model developed in the previous section the pre-requisite tests to assess the suitability of the data were done and the data set was found to be suitable.

The regression model details for 6-lane highway sections are shown in Table 5.6.

Table 5.6: Model summary and ANOVA table for 6-lane section

Model Summary[b]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Durbin-Watson
1	.927[a]	.858	.811	58.961	1.544

a. Predictors: (Constant), Effective lane width, Access point density

b. Dependent Variable: Lane Capacity (pcu/h/l)

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	126466.815	2	63233.407	18.190	.003[b]
	Residual	20858.074	6	3476.346		
	Total	147324.889	8			

a. Dependent Variable: Lane Capacity (pcu/h/l)

b. Predictors: (Constant), Effective lane width, Access point density

The regression model coefficient of determination (R^2) is 0.86 shown in Table 5.6 indicates that the independent variables account for a major portion of the variance of the dependent variable. Further, it is observed that the independent variables statistically significantly predict Lane capacity as the P value is less than 0.05.

Table 5.7: Coefficient table for 6-lane model

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.
1	(Constant)	833.835	319.243		2.612	.040
	Access point density	-23.120	8.850	-.451	-2.612	.040
	Effective lane width	363.505	99.594	.630	3.650	.011

a. Dependent Variable: Lane Capacity (pcu/h/l)

Table 5.7 portrays the coefficients of the predictor variables and their significance values which are all less than 0.05 indicating that the predictor variables can be accepted to the model.

Hence the 6-lane capacity model can be written as shown in equation (5.3),

$$C_6 = 834 + 364C_L' - 23\ C_A' \qquad (5.3)$$

Where,

C_6 = 6-Lane Capacity (pcu/h/l)

C_L' = Effective lane width (m)

C_A' = Access point density (per 400m section)

Figure 5.8 depicts the 6-lane model capacity variation with effective lane width subject to different access point (AP) density values ranging from zero to ten. The lane width has a larger impact on capacity as the change in 1m in the lane width alters the capacity by a margin of 364pcu/h/l (In comparison the HCM and IHCM guidelines state that capacity will be reduced by a factor of 167 pcu/h/l/m and 264 pcu/h/l/m). The decrease in capacity per increase in Access point is 23pcu/h/l (13 pcu/h/l in HCM). It is observed that the impact access points have on 6-lane roads is lesser than the impact they have on 4-lane roads.

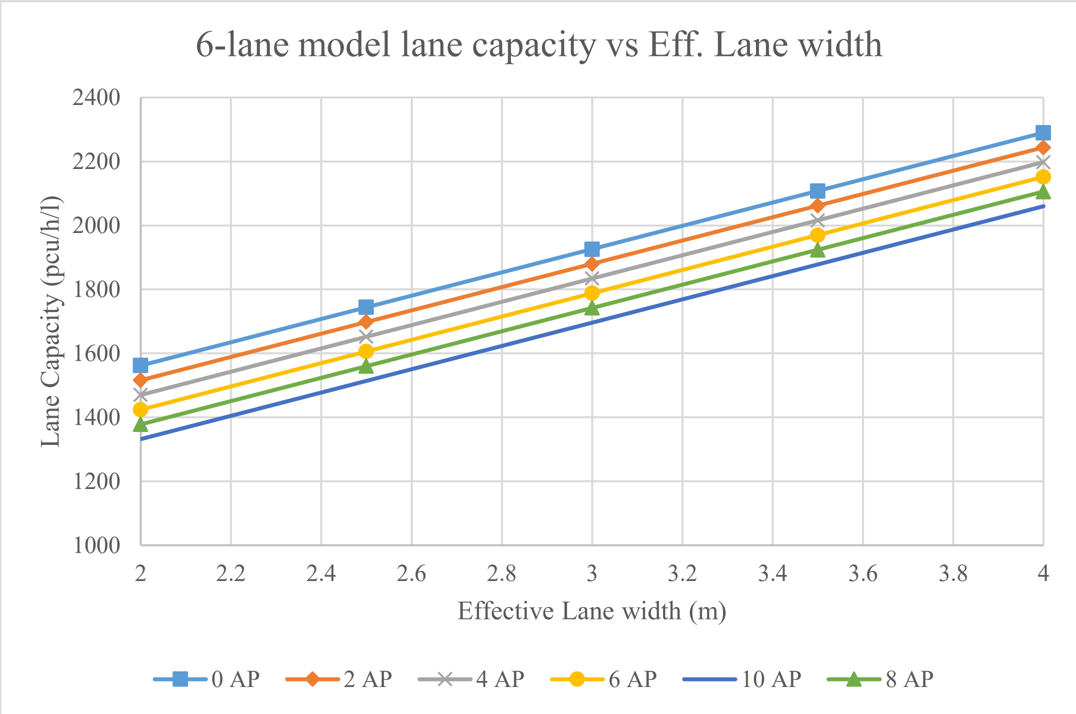


Figure 5.8: 6-lane model capacity variation with effective lane width

A summary of the 6-lane capacity model is exhibited in Table 5.8. Interpolation between values is allowed.

Table 5.8: 6-lane model capacity table

Effective Lane width (m)	Lane capacity (pcu/h/l)					
	Access Point Density					
	0	2	4	6	8	10
2.0	1562	1516	1470	1424	1378	1332
2.5	1744	1698	1652	1606	1560	1514
3.0	1926	1880	1834	1788	1742	1696
3.5	2108	2062	2016	1970	1924	1878
4.0	2290	2244	2198	2152	2106	2060

5.2.3 Combined Capacity model for multi-lane highways

The 4-lane and 6-lane models were combined in order to produce a single model to estimate capacity. The combined model is shown in equation (5.4). The coefficients present in the 4-lane and 6-lane capacity models were replaced by inserting adjustment factors for each variable. By doing so a common equation that represents both 4-lane and 6-lane models were created.

$$C_{4/6} = C_0 + C_L + C_A + C_M + C_{BE} \tag{5.4}$$

$C_{4/6}$ = Capacity (pcu/h/l)
C_0 = Capacity constant (Table 5.9)
C_L = Lane width adjustment factor (Table 5.10)
C_A = Access point density adjustment factor (Table 5.11)
C_M = Median Type adjustment factor (Table 5.12)
C_{BE} = Built environment adjustment factor (Table 5.13)

The adjustment factor tables are for each variable is presented below. Table 5.9 presents the capacity constants for 4-lane and 6-lane highways. Table 5.10, Table 5.11

, Table 5.12, Table 5.13 and depict the adjustment factors for Lane width, Access point density, Median type and Built Environment type respectively.

Table 5.9: Capacity constant factor for equation (5.4)

Highway type	Capacity constant (C_0)
4-lane highway	1467
6-lane highway	834

Table 5.10: Lane width adjustment factors for equation (5.4)

Lane width (m)	Highway type	
	4-lane	6-lane
2.0	380	728
2.5	475	910
3.0	570	1092
3.5	665	1274
4.0	760	1456

Table 5.11: Access Point Density (APD) adjustment factors for equation (5.4)

APD (per 400m)	Highway type	
	4-lane	6-lane
0	0	0
2	-78	-46
4	-156	-92
6	-234	-138
8	-312	-184
10	-390	-230
12	-468	-276

Table 5.12: Median type adjustment factors for equation (5.4)

Median Type	Highway type	
	4-lane	6-lane
Median Separated	118	0
Divided	0	0

Table 5.13: Built Environment adjustment factors for equation (5.4)

Built Environment	Highway type	
	4-lane	6-lane
Rural	0	NA
Sub-Urban	-146	0
Urban	-206	NA

5.3 Speed data

Table 5.14: Speed data of study locations

Location Code	Average stream speed (km/h)	Max stream speed (km/h)	FFS from model (km/h)	Speed at Capacity (km/h)
Loc_1	41	51	50	25
Loc_2	53	56	56	28
Loc_3	30	35	37	19
Loc_4	28	35	38	19
Loc_6	39	48	47	23
Loc_7	25	34	31	16
Loc_8	25	43	28	14
Loc_11	44	57	54	27
Loc_12	39	49	49	24
Loc_13	52	57	56	28
Loc_14	52	56	58	29
Loc_15	34	39	38	19
Loc_20	34	43	36	18
Loc_21	19	29	24	14
Loc_23	38	49	47	24
Loc_24	23	27	29	14
Loc_25	27	34	28	14
Loc_26	36	52	41	21
Loc_27	24	28	25	12
Loc_28	50	57	57	29
Loc_29	35	44	37	18
Loc_30	29	32	31	15
Loc_31	18	28	22	11
Loc_32	39	50	42	21
Loc_33	26	33	29	15
Loc_35	19	26	28	14
Loc_36	27	31	32	16
Loc_40	21	28	29	14
Loc_41	32	41	43	22
Loc_42	43	49	50	25
Loc_43	38	46	46	23
Loc_49	40	45	44	22
Loc_50	31	39	38	19
Loc_51	26	30	29	15
Loc_52	37	44	48	24
Loc_53	23	29	25	13

Table 5.14: Speed data of study locations (Continued)

Location Code	Average stream speed (km/h)	Max stream speed (km/h)	FFS from model (km/h)	Speed at Capacity (km/h)
Loc_54	28	34	33	16
Loc_57	19	38	29	15
Loc_60	14	25	19	10
Loc_65	36	43	45	22
Loc_66	30	41	34	17
Loc_68	26	40	35	18
Loc_69	24	36	33	17
Loc_73	28	35	34	17
Loc_74	28	40	38	19
Loc_75	21	26	27	13
Loc_76	22	39	34	17
Loc_78	28	44	35	18
Loc_79	36	50	46	23
Loc_80	46	52	52	26
Loc_85	47	60	53	26

5.3.1 Speeds at Capacity

Based on the speed-flow relationships derived for each location the speeds at capacity were obtained. Table 5.14 show these speeds at capacity. It was observed that the maximum speed at capacity is 29 km/h and the average is approximately 19 km/h. These are considerably low values when comparing the speeds at capacity given in the HCM guideline. Figure 5.9 shows a comparison between the two speed values. While the observed speeds at capacity were in the range of 20-30km/h the HCM speeds at capacity are in the range of 70-90 km/h.

The observed low speeds can be justified in terms of the nature of the traffic stream. In Sri Lanka the vehicles tend to ‘pack’ in the traffic stream, smaller vehicles filling gaps between the larger vehicles hence achieving higher flow values albeit at lower speeds.

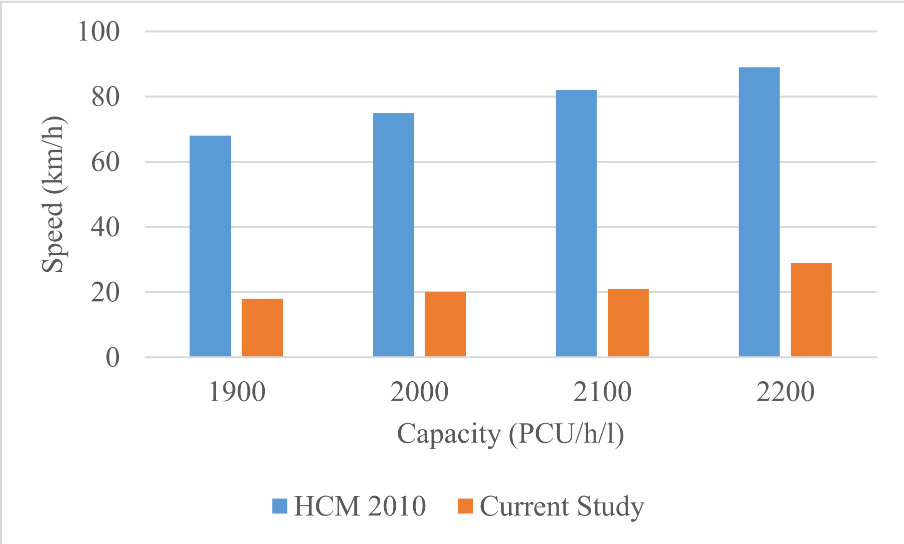


Figure 5.9: Comparison of speeds at capacity

5.3.2 Free Flow Speed Data Analysis

The maximum observed speed and the FFS (speed at 0 density from the calibrated model) were compared to examine the difference between the two data sets. The Mean Absolute Percentage Error (MAPE) formula shown by equation (5.5) was used for this purpose.

$$MAPE = \frac{1}{n}\sum_{t=1}^{n}\left|\frac{A_t - F_t}{A_t}\right| x\ 100\% \tag{5.5}$$

Where A_t is the actual value, F_t is the calculated value and n is the number of data points.

Hence, the MAPE was calculated to be 8.6% (<10%). This denotes that there isn't much difference between the two data sets. Further the Mean Absolute Error (MAE) (equation (5.6)) of the data set is 3.4 km/h. This is the average of the absolute difference between observed maximum speed and the FFS. This also shows that the calibrated model provides an acceptable fit to the observed data.

$$MAE = \frac{1}{n}\sum_{t=1}^{n}|A_t - F_t| \tag{5.6}$$

5.3.3 Free Flow Speed (FFS) estimation model

FFS is the speed of vehicles when there is no external impedance on them. Theoretically, this is the speed of a vehicle when the flow of vehicles is zero. An attempt was made to build a model to predict the FFS of the traffic stream from the roadway parameters collected during the capacity study. From this analysis it was found that the FFS dependent upon the Lateral clearance, Median type and Built Environment type. The FFS model and model development is presented in APPENDIX D.

5.4 Verification methodology of developed models

For the task of verifying developed models 10 multilane road sections were further surveyed collecting both classified flow data as well as traffic stream speed data employing the same techniques used previously.

Figure 5.10 illustrates the locations data were collected. Using the same technique followed in section 4.5 capacity values were calculated. The summary of the calculated capacities and respective FFS values is shown columns 4 and 2 respectively in Table 5.15.

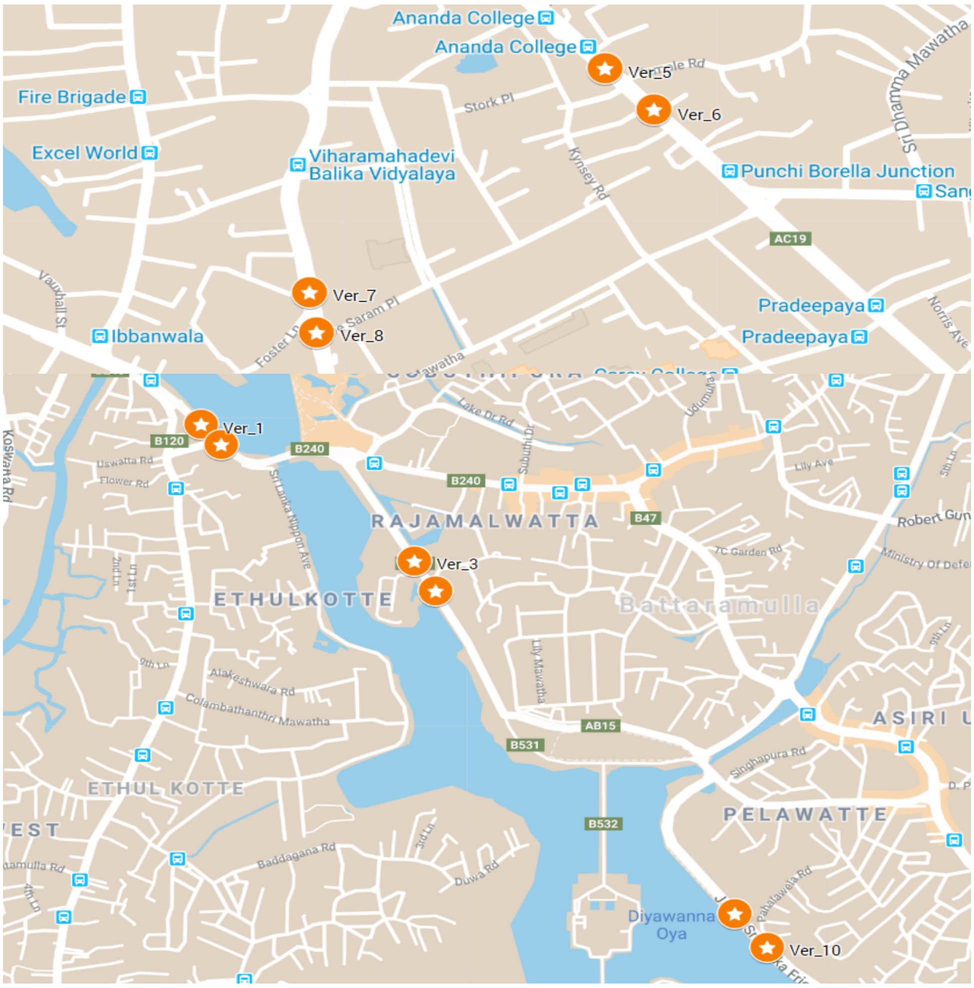


Figure 5.10: Verification data locations

Table 5.15: Developed capacity and FFS data

Location Code	FFS (km/h)	FFS from model(km/h)	Lane Capacity (pcu/h/l)	4-Lane Capacity from model(pcu/h/l)	6-Lane Capacity from model(pcu/h/l)
Ver_1	42	38.3	1666	-	1742
Ver_2	40	38.3	1753	-	1857
Ver_3	55	53	1878	2035	-
Ver_4	48	53	1972	1996	-
Ver_5	30	32.25	1450	1598	-
Ver_6	26	32.25	1455	1559	-
Ver_7	25	32.25	1543	1638	-
Ver_8	26	32.25	1455	1677	-
Ver_9	51	46.25	2169	1960	-
Ver_10	50	46.25	1785	1921	-

Using equations (5.4) and (D.1 - APPENDIX D) developed to estimate multi-lane capacity and FFS, the capacity values and FFS values of the 10 sections were calculated. The calculated values are shown in columns 3,5 and 6 in Table 5.15. A comparison of the actual and estimated capacities is shown in Figure 5.11.

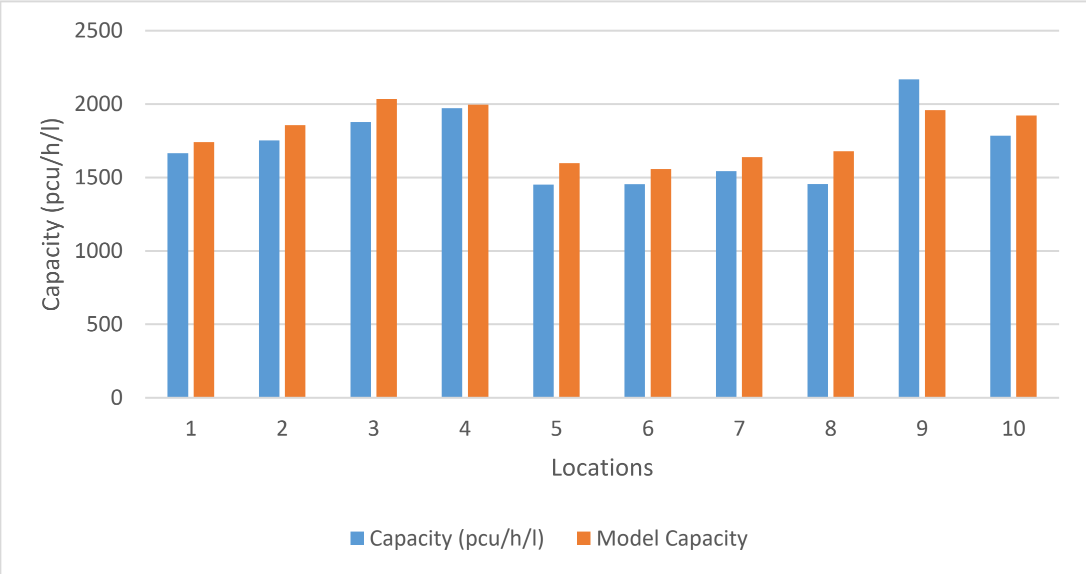


Figure 5.11: Comparison of estimated and model capacity

The Mean Absolute Percentage Error (MAPE) (equation (5.5)) of the estimated capacity is 8.2% and 5.3% (<10%) for the four-lane roads and the six-lane roads respectively. This confirms that the model accurately predicts the capacity. Further, the Mean Absolute Error (MAE) (equation (5.6)) of the data set is 128 pcu/h/l (137 pcu/h/l and 90 pcu/h/l for four-lane and six-lane roads respectively). The model shows an acceptable fit with data with R^2 value of 0.81 as seen in Figure 5.12.

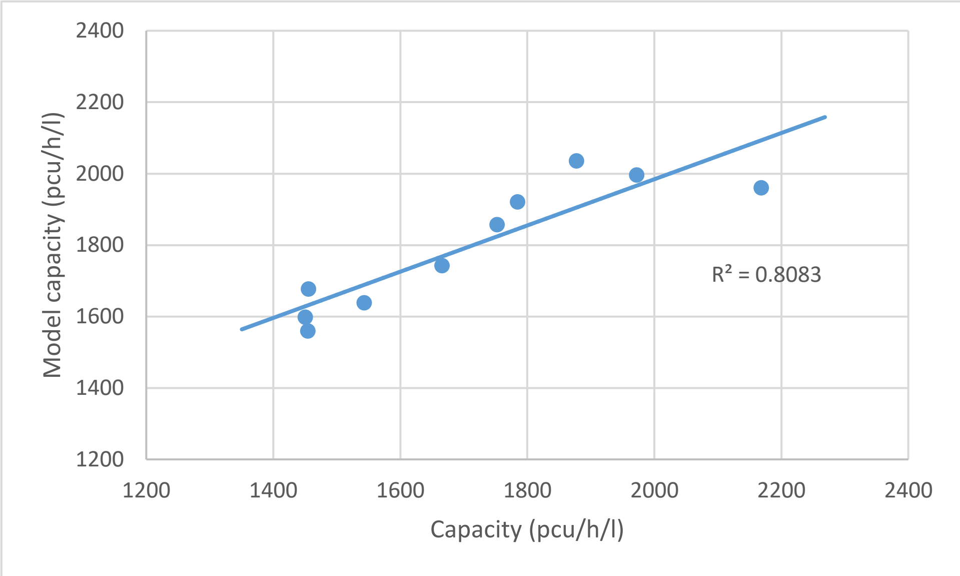


Figure 5.12: Scatter plot of model capacity and actual capacity used for validation

Similarly, a bar chart comparing the actual and model FFS values are shown in Figure 5.13.

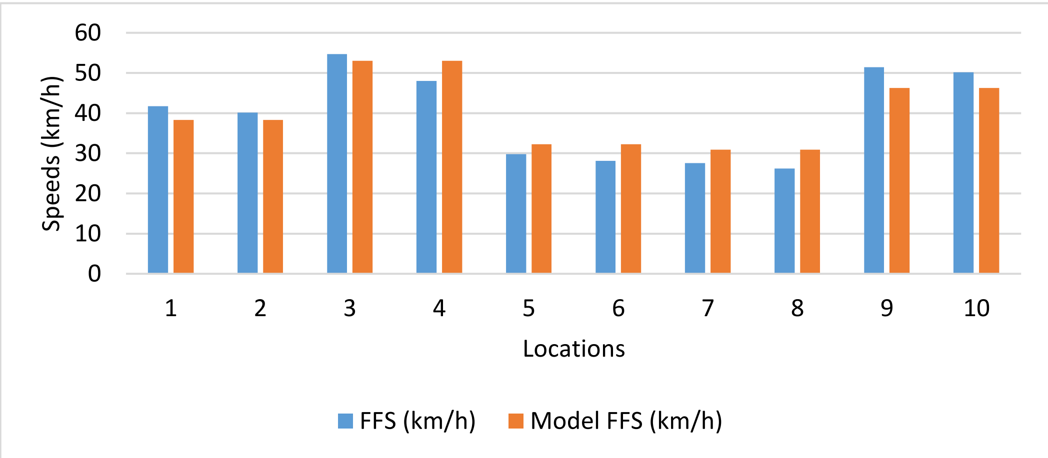


Figure 5.13: Comparison of FFS and estimated FFS from developed model

The MAPE for the estimated FFS is 9.7% (<10%) which denotes that the model accurately predicts the capacity. Further, the Mean Absolute Error (MAE) (equation (5.6)) of the data set is 3.6 km/h. The model shows an acceptable fit with data with R^2 value of 0.89 as seen in Figure 5.14.

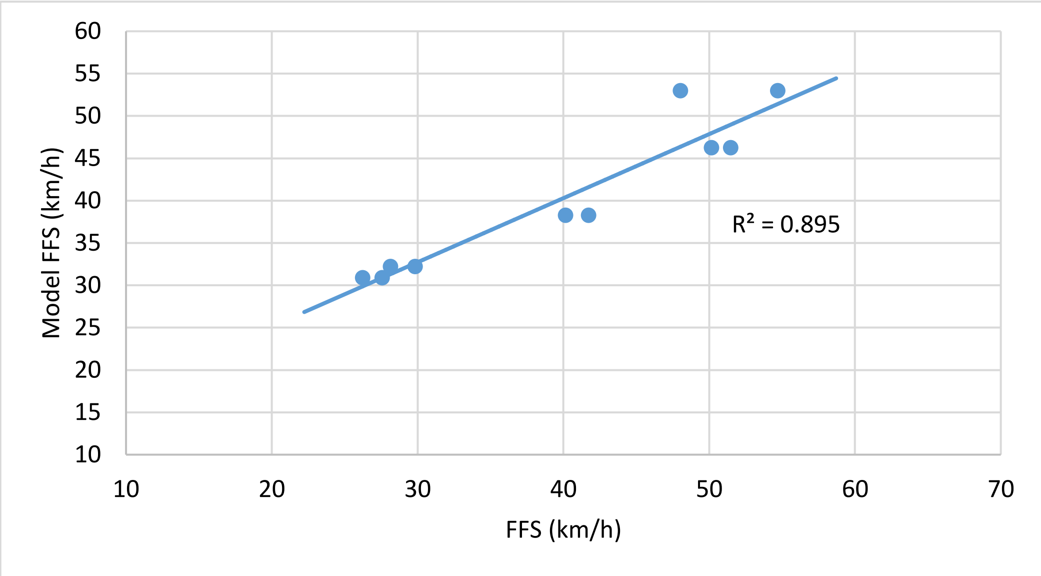


Figure 5.14: Scatter plot of model FFS and actual FFS used for validation

Based on the statistical data it is seen that the developed models accurately predict capacity as well as FFS values.

5.5 Comparison of Capacity data

Table 5.16 presents the base capacity values found in local and foreign guidelines along with base capacity values obtained through the developed models. The base capacities were estimated by keeping the effective lane width to 3.5m, Access point density zero, and the median type as median separated. This ensures that the capacity values are on par with the base conditions defined in other guidelines. When observing the developed values, it is seen that they are similar to base capacity values of other guidelines. However, the major difference is the speeds at capacity. Whilst the HCM capacity speeds are in the range of 100km/h the speeds at capacity of this study is in the range of 25 km/h. This is further discussed in section 5.3.1.

Table 5.16: Capacity comparison table

Guideline	Base Capacity (pc/h/ln)
HCM 2010	2200 (FFS = 100km/h)
IHCM	1650
AustRoads	2200
RDA Guideline	2000 (HCM 1985)
4-lane capacity	2250 (Rural) 2106 (Sub-Urban) 2044 (Urban)
6-lane capacity	2108 (Sub-Urban)

A comparative study was done to evaluate the IHCM 1997 capacity estimation model and the capacity estimation models proposed by Semeida [40]. The data used for the model verification is utilized for this purpose. The equations (2.2), (2.3) were used to derive the IHCM capacity values and equations (2.12), (2.13), (2.14) were used to derive capacity values from models proposed by Semeida. It is observed that the IHCM 1997 model underestimated capacity whereas the model proposed by Semeida both under and overestimated the capacity values by significant margins. These are confirmed by the R^2 values of 0.48 and 0.48 and high RMSE values of 305.60 and 389.80 for the IHCM model and Semeida model respectively.

Table 5.17: Capacity values estimated from different models in literature

Location	Actual Capacity (pcu/h/l)	This Study (pcu/h/l)	IHCM 1997 (pcu/h/l)	Semeida 2013 (pcu/h/l)
Ver_1	1666	1742	1494	1880
Ver_2	1753	1857	1494	1880
Ver_3	1878	2035	1606	1600
Ver_4	1972	1996	1606	1598
Ver_5	1450	1598	1397	NA
Ver_6	1455	1559	1397	NA
Ver_7	1543	1638	1397	NA
Ver_8	1455	1677	1397	NA
Ver_9	2169	1960	1465	1448
Ver_10	1785	1921	1465	1449
R^2	**-**	**0.81**	**0.48**	**0.48**
RMSE	**-**	**139.61**	**305.60**	**389.80**

5.6 Limitations of Study

The study was carried out within a set of restrictions, most of which were linked to the amount of data available for analysis. Following are the limitations observed in the study,

- The vehicle composition was not considered as a factor for the developed models and the vehicle composition was assumed to be uniform among all study locations. Figure 5.15 shows a pie chart of the split of vehicles surveyed during the study. It is observed that 52% of the vehicles are small vehicles (Three-wheelers and Motorcycles). Table 5.18 shows a complete breakdown of the percentages of all vehicles along with the standard deviation in each category. The highest deviation from the mean is observed in the 'Car' category with a value of 8%. The deviations from the mean are considered to be insignificant during this study.

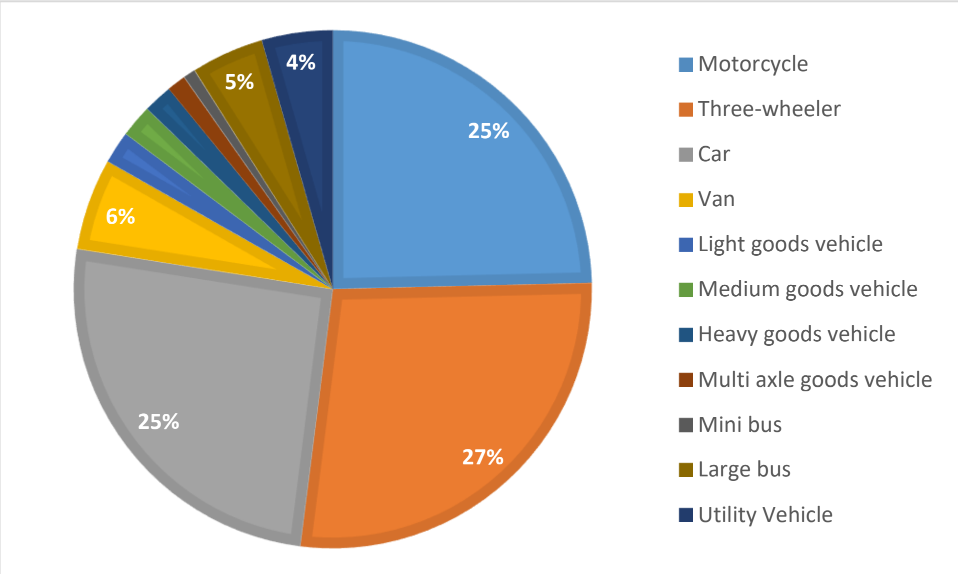


Figure 5.15: Overall vehicle composition of surveyed locations

Table 5.18: Overall vehicle composition and standard deviation

Vehicle type	Percentage of vehicles	Standard Deviation
Motorcycle	24%	6%
Three-wheeler	27%	5%
Car	25%	8%
Van	6%	2%
Utility Vehicle	4%	2%
Light goods vehicle	2%	1%
Medium goods vehicle	2%	1%
Heavy goods vehicle	2%	1%
Multi axle goods vehicle	1%	1%
Minibus	1%	0%
Large bus	4%	2%

- The input variables of the study is within the following limits shown below. Extrapolating may not prudent given that no information available to support it.
    - Effective Lane width: 2.1m – 4.0m
    - Access Point Density: 0 – 13 (per 400m)
    - Lateral Clearance: 0m – 4m

- There are other roadway factors that may affect lane capacity such as pedestrian activity, traffic composition, parking movements etc. that are not analyzed in this study.

6 CONCLUSIONS AND RECOMMENDATIONS

It is understood that capacity is a vital parameter in transport planning and traffic management. Hence many transport authorities around the world have developed guidelines to evaluate capacity. Since the traffic carrying capacity of a road varies with new developments in vehicle technology, road construction etc. the capacity value needs to be re-evaluated periodically. Further, since it has been established that capacity is a parameter that depends upon roadway and traffic characteristics proper methodologies should be in place to accurately estimate capacity. Presently this is not the case in Sri Lanka.

Hence this research was designed with the aim to develop a model which can predict the capacity of multi-lane roads in Sri Lanka. This aim is sub divided into three main objectives which are to find if the US HCM 2010 multi-lane methodology was applicable to Sri Lankan conditions, to investigate the impact various factors have on multi-lane capacity, and to finally develop a model based on these characteristics. Section 3.1 discusses the study done to test the applicability of the HCM 2010 methodology to Sri Lankan conditions and the finding was that it is not applicable given the low traffic stream speeds observed on local roads. The HCM methodology does not capture the heterogenous nature of the local traffic nor the unique roadside conditions observed in Sri Lanka. Section 4 discusses the different factors such as lane width, built environment, access point density that influence the traffic carrying capacity of roads and section 5.2 discusses the developed models to estimate capacity based on the identified factors.

Considering the factors that affect multi-lane capacity, it was observed that the effective lane width, access point density, median separation, and roadside built environment has a significant impact on lane capacity. The factors that did not have a statistically significant impact were Shoulder type, lateral clearance, and FFS. The 6-lane capacity model which was developed from a limited data set which included only median separated road sections in sub-urban environments was based on effective lane width and access point density. This area of the study can be strengthened by further research.

The approach followed to establish capacity is based on Greenshields' model and first principles of traffic engineering. This method is simple and has been proven through numerous studies to be a reliable method to estimate capacity. Section 2.6 in the literature review discusses alternative methods available to estimate capacity. But these methods are either not universally applicable or extremely data intensive (eg. Van Aerde method etc.). Further, considering the factors that capacity is estimated in this study, the focus is mainly on the roadway characteristics. Whilst in some studies it has been observed that FFS is a good predictor of capacity this was not the case on Sri Lankan roads. Further research can be done into areas such as vehicle composition and its effect on highway capacity.

The typical base capacity for a 4-lane urban road was found to be 2044 pcu/h/l. The base capacities for 4-lane rural and sub-urban sections were estimated to be 2250 pcu/h/l and 2106 pcu/h/l respectively. Further, the base capacity for a 6-lane sub-urban road section was estimated to be 2108 pcu/h/l. These values are comparable in magnitude to base capacity values found in literature (HCM 1900-2200pcu/h/l; RDA 2000pcu/h/l etc.). But the traffic stream condition at which these values are obtained are quite different as the traffic stream speeds are in the range of 20-30km/h as opposed to being in the range of 70-100km/h expected in the HCM 2010 manual. Hence these capacity states are unstable as they are susceptible to break down causing congestion.

Additionally, a model to estimate FFS was also developed using the collected data. The predictor variables for this model included the built environment, lateral clearance and the median type of the road. The typical FFS of a rural road section with 2m lateral clearance and a center median was 50km/h. Sub-urban and urban road sections with similar conditions have 36km/h and 35km/h FFS speeds respectively. This shows the state of the traffic stream speed on Sri Lankan multi-lane roads.

The developed models were verified with another data set to check the accuracy of prediction. It was seen that the models accurately predicted the capacity and FFS values with R^2 values 0.81 and 0.89. Capacity, FFS data tables were developed for ease of use and are presented under Table 5.5, Table 5.8, and Table D-3**Error! Reference source not found.**. These can be used for traffic engineering studies related to multi-lane roads.

New techniques for traffic data collection were tested and used for this research study. Flow and speed collection methods such as the TRAZER software, TIRTL instrument, and crowdsourced google speed data were used to collect data. The advantages and drawbacks of these methods were studied and discussed in APPENDIX A.

Finally, the outcomes of the research study can be used in the development of a local guideline for capacity estimation as it is important given the incompatibility of foreign guidelines. Further studies can be done into other criteria that may influence capacity in future studies using these outcomes as a foundation.

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APPENDIX A: Comparative Study of Data Collection Techniques

A.1 Study Locations

For this comparative study data were collected at three different locations. The methods used for data collection at each location and the geometric parameters of the locations are shown in Table A- 1.

Table A- 1: Study locations

	Road name	Location	GPS coordinates	No. of lanes	Lane width (m)	Section Cross-fall	Survey methods used
P1	A4 - Highlevel Road	Pannipitiya	6°50'41.9"N 79°57'15.4"E	4	3.35	Normal Camber	TIRTL, TRAZER, Videography
P2	AB11 - New Galle Road	Panadura	6°43'41.9"N 79°53'54.2"E	4	3.4	Super Elevated	TIRTL, Videography
P3	A8 - Horana Road	Panadura	6°42'24.1"N 79°56'28.4"E	2	3.5	Normal Camber	TIRTL, Videography

The locations (shown in Figure A- 1) were selected such that road sections with both normal camber and super-elevations are covered. Since the best method to get an accurate count is by analyzing a video, videos were recorded at all survey locations and manual counts were done using the videos. These counts were considered as base values for comparison.

Figure A- 1: Data collection locations

A.2 Results and Discussion – Flow

The error was calculated using Equation (6.1). True count is the manual count done by reviewing the videos captured at each location. The output counts from the automated methods are compared with this value.

$$Error = \frac{True\ count - X}{True\ count}\ x\ 100\%;\ \text{where}\ x = TIRTL\ count \qquad (6.1)$$

A.2.1 TIRTL Instrument Flow Analysis

Table A- 2: TIRTL flow summary

Location	P1		P2		P3	
	Count	% Error	Count	% Error	Count	% Error
TIRTL	2994	-15%	4146	-2%	5121	-4%
True Count	3529		4210		5325	

As observed in Table A- 2 the flow count values produced in each location have varying error percentages. Locations P2 and P3 have low error values which are acceptable but location P1 has a high error in the total count. These errors can be explained with respect to the geometry of each location and the data collection method of the instrument. The road geometries of the locations P1, P2, and P3 are as listed below and illustrated in Figure A- 3,

P1 – Four-lane road (normal cross-fall)
P2 – Four-lane road (super-elevated section)
P3 – Two-lane road (normal cross-fall)

The range of the TIRTL instrument is as shown in Figure A- 2. The optimal height for the infra-red beam is 60mm above the road level with a tolerance of -25mm to +35mm.

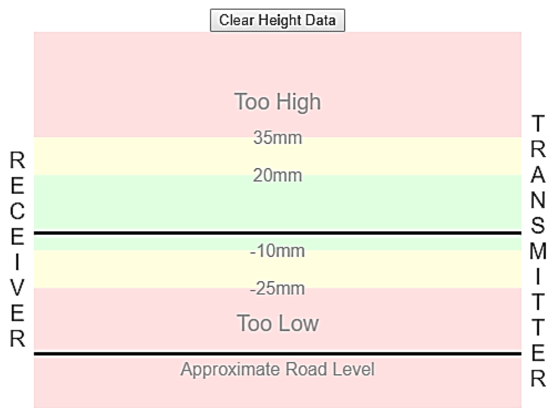

Figure A- 2: The range of the TIRTL instrument

Table A- 3: Error % per lane in TIRTL – Location P1

Location – P1	lane1	lane 2	lane 3	lane 4	Total
Actual count	855	829	945	900	3529
TIRTL	704	720	845	725	2994
Error	-18%	-13%	-11%	-19%	-15%

As observed in Table A- 3 the errors in lanes 1 and 4 (outer lanes) are higher compared to those of the inner lanes. This is because the height between the TIRTL instrument IR beam and the road surface is higher than the recommended range. This is illustrated in Figure A- 3 (not to scale).

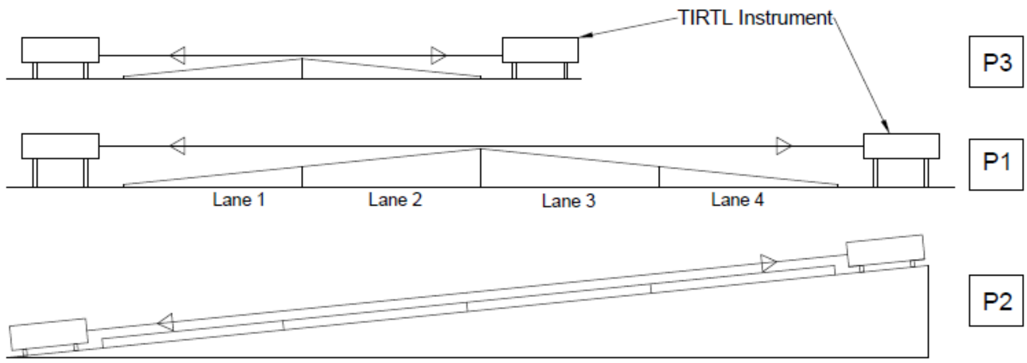

Figure A- 3: Loci of Infra-red beams - TIRTL

A similar issue as in location P1 was encountered at location P3 but due to the shorter carriageway width the vertical rise of the road is lesser. Hence an error of only -4% was observed in the results at that location. The error in vehicle count estimate was minimum at location P2 since there was no cross-fall at that section.

Hence it was observed that the TIRTL instrument was suitable for roads with short carriageway width (two lane roads) or for multi-lane roads with no camber. Another work around for this issue is to place the receiver unit next to the center median of the road. But this may cause disruptions in the traffic flow.

A.2.2 TRAZER Software Flow Analysis

As shown in Table A- 4, 3529 vehicles were analyzed at location P1 using the TRAZER software. Initially the traffic flow was captured as specified in the user manual. Once flow video is uploaded to the software the internal analysis procedure of TRAZER has 4 main steps.

Step 1: Inputting of geometric and vehicle class dimensions to the software and processing flow video.

Step 2: Reviewing of identified vehicles and deleting false vehicle recognitions (see Figure A- 4).

Step 3: Reviewing identified vehicles and confirming/classifying vehicles to the correct vehicle class. In this step vehicles that are identified but are in the wrong class are moved to the correct one. Also, in the software reviewed for this research HMV's are not classified as BUS and TRUCK by the software. Therefore, manual classification of these vehicles is done in this step.

Step 4: Addition of unidentified vehicles by reviewing the video using TRAZER software.

As seen in Table A- 4 the estimate provided by the TRAZER software after step 1 is incorrect by a margin of 843 vehicles. The error is calculated using Equation (6.2),

$$Error = (True\ count - X)/(True\ count)\ x\ 100\% \qquad (6.2)$$

where X = Count of step 1, step 2, step 3

Hence an error of +24% was observed after the step 1. On further inspection it was observed that LMV, 2W categories were overestimated by the software whereas 3W and HMV categories are underestimated. Of the 2W count of 1303, only 579 were accurate identifications, 724 being false positives and incorrect classifications. This was a major factor that affected the initial estimate of flow. It was observed that vehicle side mirrors are identified by the software as 2W's leading to this error. This is phenomena is seen in the Figure A- 4.

Table A- 4: Summary of TRAZER results

TRAZER Analysis	LMV		3W		HMV		BUS		TRUCK		2W		**Total**	
	Count	Error	Count	Error	Count	Error	Count	Error	Count	Error	Count	Error	**Count**	**Error**
Step 1: Process	2260	31%	573	-14%	236	-33%	0		0		1303	65%	**4372**	**24%**
Step 2: Deletion	1540	-10%	497	-25%	147	-58%	0		0		623	-21%	**2807**	**-20%**
Step 3: Reclassification	1500	-13%	544	-18%	184	-48%	66	-63%	118	-33%	579	-27%	**2807**	**-20%**
Step 4/True Count	1720		667		352		177		175		790		**3529**	

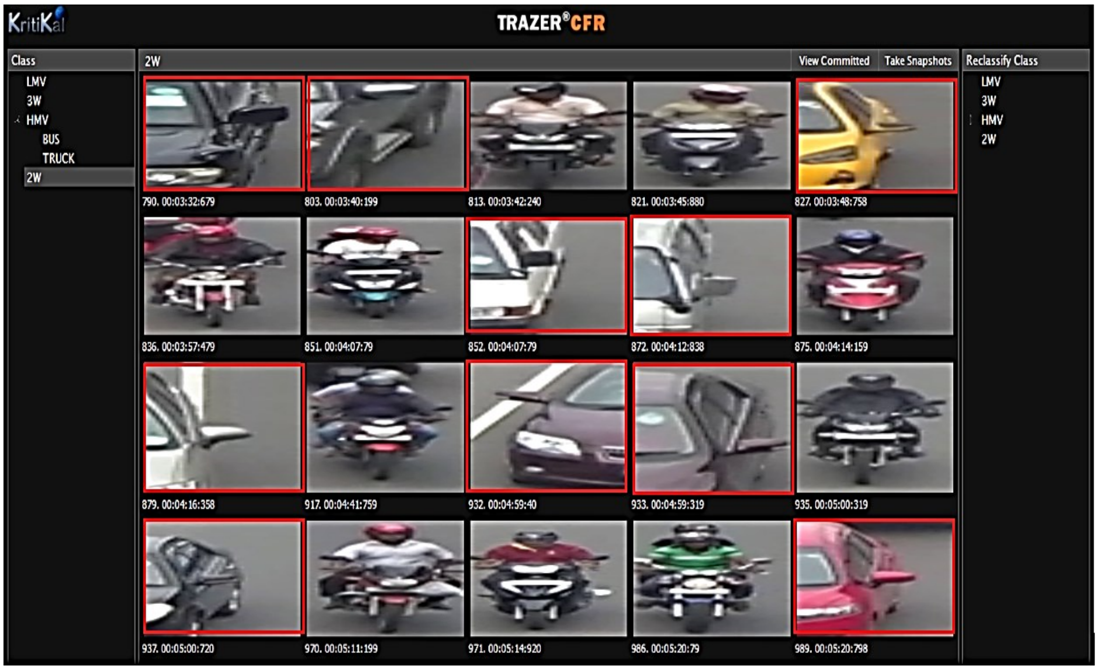

Figure A- 4: Error with side mirrors in TRAZER

After steps 2 and 3 (deletion and reclassification) the total vehicle count estimated by the software is 20% less than the actual value. Of the individual categories HMV and 2W categories were off by -48% and -27% respectively. Hence it was observed that the software is less capable at identifying vehicles with non-standard/irregular dimensions (Large and small). The estimate of LMV's were at an acceptable level of 87%. The accuracy of the estimates of 2W, 3W and HMV's were 73%, 82% and 52% respectively.

The final step is the addition of unidentified vehicles manually. This is a tedious and time intensive process as the video needs to be analyzed frame by frame to detect vehicles that have not been identified by the software. However, at the end of this process 100% accuracy can be achieved.

<u>Estimation capability with flow</u>

TRAZER software's ability to detect vehicles with changes in flow is analysed in this section. shows the 1 min flow values during the survey period.

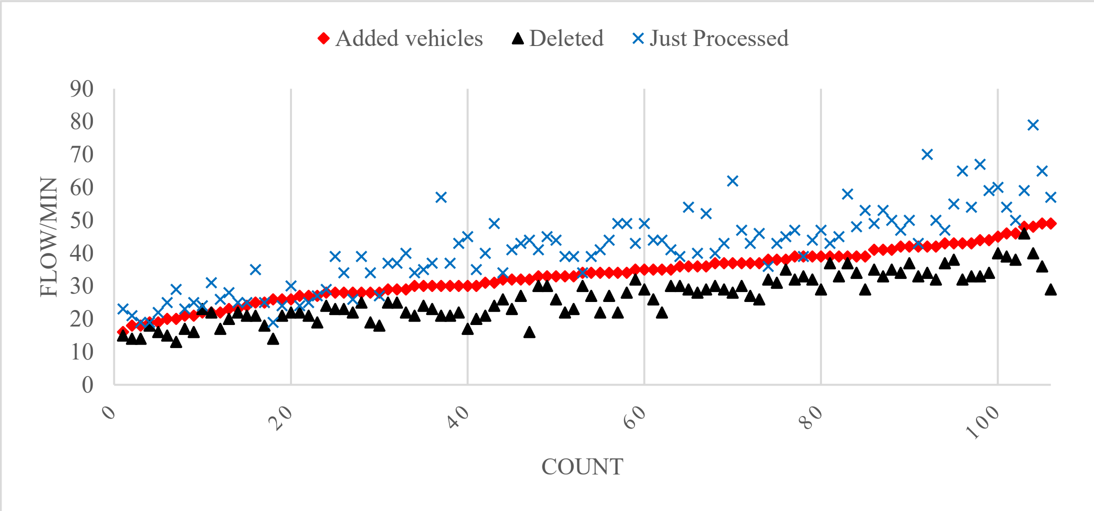

Figure A- 5: TRAZER flow values

Directional flowrates varying from 960 veh/h/dir to 2940 veh/h/dir were observed during the study. Since the surveyed road was a four-lane highway the highest and lowest lane flowrates were 480 veh/h/l and 1470 veh/h/l.

To check if the traffic flow influenced the vehicle counting ability of the TRAZER software two samples of 25 one-minute flows were tested. A two-sample t-test assuming unequal variances was performed on the difference in flows after step 2 of the processing sequence in the software. Table A- 5 depicts the lowest and highest 25 flow values (veh/min) and the difference in flows to which the t-test was done.

H_0: The is no difference in the means of the two data sets
H_1: There is a difference in the means of the two data sets
Table A- 6 shows the sample means and variances. The p-value of the t-test is 0.00018 at 95% significance. Hence the null hypothesis is rejected meaning that there is a significant difference between the means of the two samples. Therefore, it can be said that the magnitude of the flow has an effect on the accuracy of the TRAZER software flow prediction.

Table A- 5: Lowest and highest flows observed during study

Sample 1				Sample 2			
Index	**True count (veh/min)**	**After step 2 (deletion) (veh/min)**	**Difference in flows (veh/min)**	**Index**	**True count (veh/min)**	**After step 2 (deletion) (veh/min)**	**Difference in flows (veh/min)**
1.	16	15	**1**	*82.*	39	33	**6**
2.	18	14	**4**	*83.*	39	37	**2**
3.	18	14	**4**	*84.*	39	34	**5**
4.	19	18	**1**	*85.*	39	29	**10**
5.	19	16	**3**	*86.*	41	35	**6**
6.	20	15	**5**	*87.*	41	33	**8**
7.	20	13	**7**	*88.*	41	35	**6**
8.	21	17	**4**	*89.*	42	34	**8**
9.	21	16	**5**	*90.*	42	37	**5**
10.	22	23	**-1**	*91.*	42	33	**9**
11.	22	22	**0**	*92.*	42	34	**8**
12.	22	17	**5**	*93*	42	32	**10**
13.	23	20	**3**	*94.*	43	37	**6**
14.	23	22	**1**	*95.*	43	38	**5**
15.	24	21	**3**	*96.*	43	32	**11**
16.	25	21	**4**	*97.*	43	33	**10**
17.	25	18	**7**	*98.*	44	33	**11**
18.	26	14	**12**	*99.*	44	34	**10**
19.	26	21	**5**	*100.*	45	40	**5**
20.	26	22	**4**	*101.*	46	39	**7**
21.	27	22	**5**	*102.*	46	38	**8**
22.	27	21	**6**	*103.*	48	46	**2**
23.	27	19	**8**	*104.*	48	40	**8**
24.	28	24	**4**	*105.*	49	36	**13**
25.	28	23	**5**	*106.*	49	29	**20**

Table A- 6: Sample means and variances

	Sample 1	*Sample 2*
Mean	4.2	7.96
Variance	7.4	13.7

A.3 Results and Discussion – Speed

A.3.1 TIRTL Instrument Speed Analysis

The TIRTL was deployed for data collection at all three survey locations. The speed data collected were compared with speed data computed manually. The manual speed data calculation was done by observing the video and calculating the time travelled for vehicles to traverse a known distance [3]. A sample of 177 vehicles were selected for the speed survey. As shown in Table A- 7 the Mean Absolute Error (MAE) is 3.47, the Root Mean Square Error (RMSE) is 4.65 and the Mean Absolute Percentage Error (MAPE) is 8.9% which are acceptable values denoting that instrument was able to capture the speeds of individual vehicles with high accuracy.

Table A- 7: Statistical data of TIRTL speed survey

Max Speed	81 km/h
Min Speed	12 km/h
Average Speed	42 km/h
MAE	3.47
RMSE	4.65
MAPE	8.9%

Figure A- 6 depicts the error terms of each speed estimation. The average error in speed was 3.47 km/h.

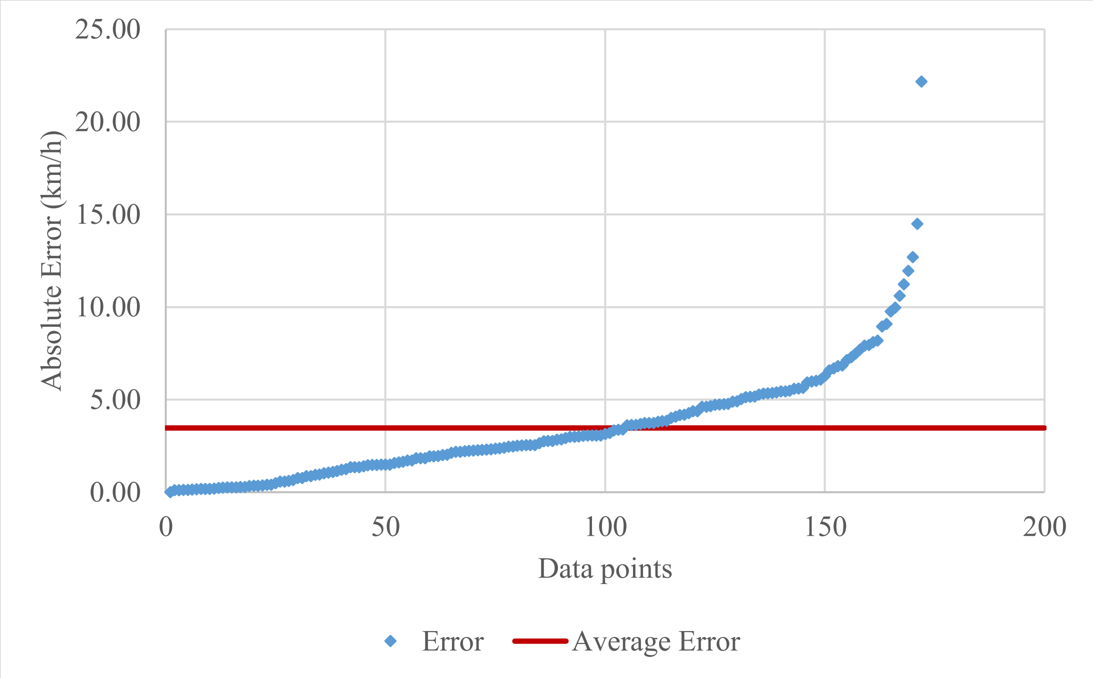

Figure A- 6: TIRTL speed error values

A.3.2 TRAZER software Speed Analysis

Traffic speed data of a selected group of vehicles were collected by analysing the captured videos using TRAZER software. The data was compared with the corresponding actual speed data to evaluate the accuracy of the outputs of TRAZER using a similar methodology as used in the TIRTL speed analysis. The statistical data of the study are given in Table A- 8. According to the results of the study it is seen that the TRAZER software predicts the speeds of vehicles at a higher accuracy than the TIRTL instrument.

Table A- 8: Statistical data of TRAZER software speed survey

Max Speed	50 km/h
Min Speed	19 km/h
Average Speed	35 km/h
MAE	2.57
RMSE	3.31
MAPE	2.6%

A.3.3 Comparison of Speed Estimation of TIRTL & TRAZER

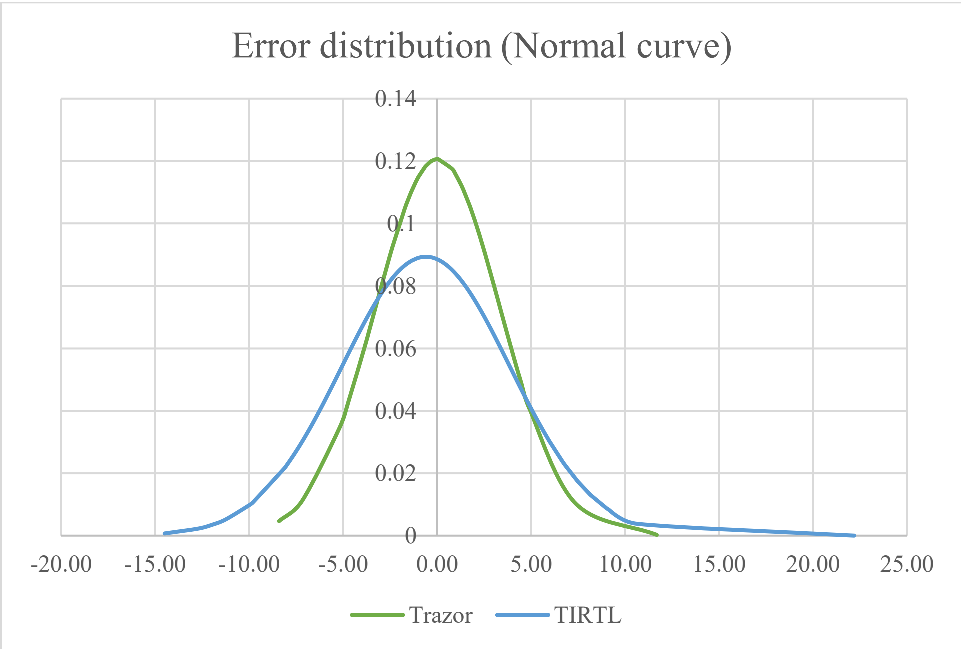

Figure A- 7: Error distribution in TIRTL and TRAZER

Comparing the two automated speed detection methods it was observed that the TRAZER had a comparatively smaller spread in error in speed detection. This can be seen in the graph in Figure A- 7. The TIRTL instrument had a RMSE of 4.65 whereas TRAZER had a RMSE of 3.31. Hence the TRAZER software is the better predictor of speeds when comparing the two methods.

A.4 Google Distance Matrix (GDM) API Speed

A 550m straight section was selected along AB11 road between Moratuwa and Panadura as shown in Figure A- 8. The parameters shown in Table A- 9: Parameters available in Google Distance Matrix API were inputted to the script developed by Sakitha et al [72] for travel time data collection using Google Distance matrix API. This script enables the user to input the road catchment coordinates in terms of GPS coordinates for each direction shown in Table A- 10 Table A- 10: GPS coordinates of survey locationsof travel and retrieve travel time data at a desired time interval. The script was scheduled to be called using a website that provides scheduled task services. For this study a time interval of 1 min was chosen between each data collection.

Table A- 9: Parameters available in Google Distance Matrix API

Parameter	Options available	Selected Option
Travel mode	Driving, Walking, Bicycling, Transit	Driving
Traffic model	Pessimistic, Optimistic, Best Guess	Best Guess

The use of ‘Driving’ travel mode ensures that the distance calculation is done along the road network. Walking requests distance calculation for walking via pedestrian paths & sidewalks (where available). Bicycling requests distance calculation for bicycling via bicycle paths & preferred streets (where available). Transit requests distance calculation via public transit routes (where available). Similarly, the traffic model – Best Guess indicates that the returned ‘duration_in_traffic’ should be the best estimate of travel time given what is known about both historical traffic conditions and live traffic. Live traffic becomes more important the closer the ‘departure_time’ is to now.

Table A- 10: GPS coordinates of survey locations

Direction	Start GPS coordinates	End GPS coordinates
Moratuwa to Panadura	6.731302, 79.896373	6.727760, 79.899085
Panadura to Moratuwa	6.727693, 79.899055	6.731275, 79.896291

A manual speed survey using videography was carried out for over two hours in each direction parallel to the Google Distance matrix API and the two speeds were compared. The speed data shown are aggregated 5-minute interval values. These were computed by getting the average speed of vehicles within a 5-minute interval and the average speed of the 1 – minute interval Google Distance matrix data.

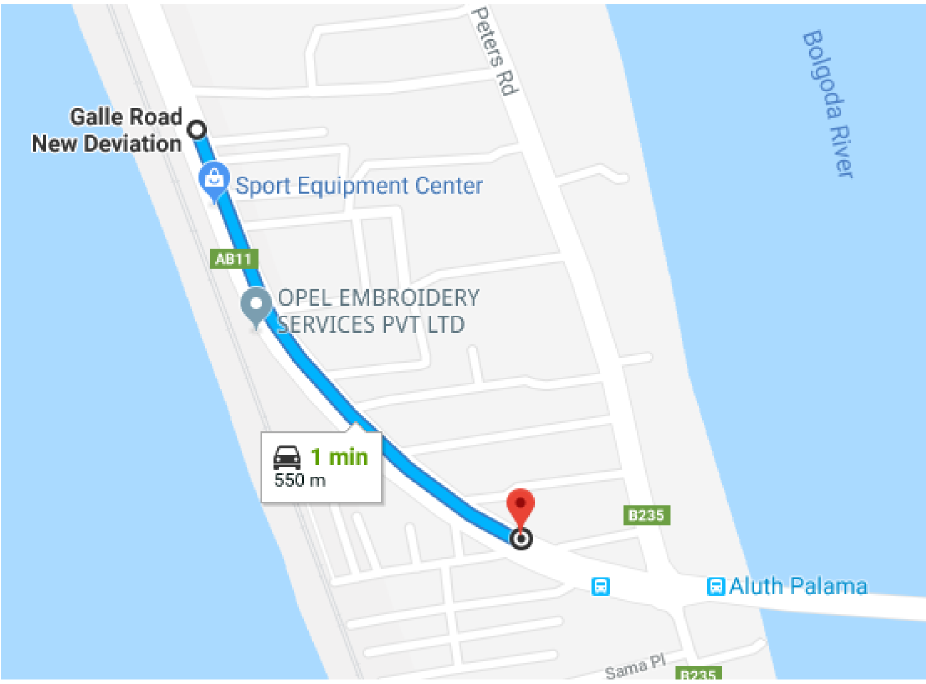

Figure A- 8: Road section selected for study [source: Google maps]

Considering the statistical data shown in Table A- 11 it is observed that the speeds predicted are of high accuracy given that the RMSE value is 0.97 and the MAPE value is 1.7%.

Table A- 11: Statistical data of Google Distance Matrix speed survey

Max Speed	53km/h
Min Speed	47km/h
Average Speed	50km/h
MAE	0.87
RMSE	0.97
MAPE	1.7%

The difference between the speed estimation using GDM API and the other two techniques is that GDM provides traffic stream speeds as opposed to individual vehicle speeds. Hence the ability evaluate individual vehicle speeds isn't available through this method. But since the research requirement is traffic stream speed data this was a viable method.

APPENDIX B: Traffic Flow Counts

Speed and flow data of a selected location is shown herewith.

Speed data of Location ID: Loc_41 (Table B- 1)

Flow data sheet of location ID: Loc_41 (Table B- 2)

Table B- 1: Speed data of location: Loc_41

Time	Speed (km/h)	Time	Speed (km/h)	Time	Speed (km/h)	Time	Speed (km/h)
06.00 - 06.15	40.5	10.00 - 10.15	31.8	14.00 - 14.15	36.4	18.00 - 18.15	34.8
06.15 - 06.30	35.2	10.15 - 10.30	32.6	14.15 - 14.30	35.7	18.15 - 18.30	34.7
06.30 - 06.45	28.1	10.30 - 10.45	31.5	14.30 - 14.45	34.0	18.30 - 18.45	34.4
06.45 - 07.00	21.4	10.45 - 11.00	32.3	14.45 - 15.00	35.0	18.45 - 19.00	34.6
07.00 - 07.15	24.1	11.00 - 11.15	31.7	15.00 - 15.15	35.2	19.00 - 19.15	34.6
07.15 - 07.30	26.1	11.15 - 11.30	31.2	15.15 - 15.30	32.9	19.15 - 19.30	34.0
07.30 - 07.45	22.9	11.30 - 11.45	31.2	15.30 - 15.45	34.3	19.30 - 19.45	34.5
07.45 - 08.00	19.3	11.45 - 12.00	31.5	15.45 - 16.00	33.4	19.45 - 20.00	34.5
08.00 - 08.15	15.4	12.00 - 12.15	33.6	16.00 - 16.15	34.1	20.00 - 20.15	34.3
08.15 - 08.30	14.6	12.15 - 12.30	35.2	16.15 - 16.30	34.6	20.15 - 20.30	34.2
08.30 - 08.45	17.4	12.30 - 12.45	33.2	16.30 - 16.45	34.9	20.30 - 20.45	35.0
08.45 - 09.00	25.0	12.45 - 13.00	32.5	16.45 - 17.00	36.0	20.45 - 21.00	36.2
09.00 - 09.15	28.7	13.00 - 13.15	33.7	17.00 - 17.15	34.2	21.00 - 21.15	36.5
09.15 - 09.30	31.0	13.15 - 13.30	34.9	17.15 - 17.30	35.0	21.15 - 21.30	37.2
09.30 - 09.45	32.2	13.30 - 13.45	37.5	17.30 - 17.45	35.2	21.30 - 21.45	37.8
09.45 - 10.00	32.2	13.45 - 14.00	37.8	17.45 - 18.00	35.5	21.45 - 22.00	37.7

Table B- 2: Flow data of location: Loc_41

Time	Motorcycle	Three-wheeler	Car	Van	Utility Vehicle	Light Goods Vehicle	Medium Goods Vehicle	Heavy Goods Vehicle	Multi Axle Vehicle	Mini-Bus	Large Bus	Total
06.00 - 06.15												
06.15 - 06.30												
06.30 - 06.45	253	180	219	43	34	6	19	3	1	9	39	806
06.45 - 07.00	285	172	234	47	32	7	17	0	1	9	41	845
07.00 - 07.15	294	199	256	56	33	5	16	1	1	8	43	912
07.15 - 07.30	288	158	233	36	26	6	9	1	1	6	24	788
07.30 - 07.45	394	188	307	31	27	5	10	2	1	1	22	988
07.45 - 08.00	452	226	248	24	28	8	8	1	0	1	26	1022
08.00 - 08.15	438	222	311	38	37	2	24	4	2	3	25	1106
08.15 - 08.30	361	218	226	30	34	19	31	11	1	2	16	949
08.30 - 08.45	421	181	217	33	31	15	20	12	5	3	33	971
08.45 - 09.00	356	193	154	25	29	24	37	15	6	0	34	873
09.00 - 09.15	193	182	98	19	16	13	27	18	5	0	48	619
09.15 - 09.30	198	178	51	28	19	18	29	27	6	1	31	586
09.30 - 09.45	199	171	104	40	30	27	32	33	8	2	18	664
09.45 - 10.00	190	188	76	23	26	14	49	28	5	1	30	630
10.00 - 10.15	191	158	90	20	27	7	31	26	27	0	19	596
10.15 - 10.30	185	139	67	29	22	15	36	17	17	0	20	547
10.30 - 10.45	132	98	59	21	19	8	37	32	16	1	17	440
10.45 - 11.00	123	128	46	20	20	20	29	16	4	2	27	435
11.00 - 11.15	141	139	76	22	24	19	40	12	7	0	17	497

Table B- 2: Flow data of location: Loc_41 (Continued)

Time	Motorcycle	Three-wheeler	Car	Van	Utility Vehicle	Light Goods Vehicle	Medium Goods Vehicle	Heavy Goods Vehicle	Multi Axle Vehicle	Mini Bus	Large Bus	Total
11.15 - 11.30	127	144	78	20	23	28	52	17	6	0	23	518
11.30 - 11.45	113	136	78	23	19	13	44	12	13	1	18	470
11.45 - 12.00	144	138	61	28	14	19	45	18	23	1	11	502
12.00 - 12.15	82	146	40	16	20	16	31	14	10	0	8	383
12.15 - 12.30	85	114	47	19	19	15	28	12	11	0	7	357
12.30 - 12.45	90	97	96	39	21	19	51	23	17	1	19	473
12.45 - 13.00	99	143	59	23	28	9	40	11	28	0	22	462
13.00 - 13.15	114	103	61	31	26	16	39	20	16	0	23	449
13.15 - 13.30	116	108	46	35	25	10	41	17	13	1	20	432
13.30 - 13.45	94	123	41	28	27	15	28	27	12	0	14	409
13.45 - 14.00	87	98	50	18	18	17	32	22	11	1	16	370
14.00 - 14.15	64	109	60	29	17	16	41	24	17	0	19	396
14.15 - 14.30	79	97	57	27	19	21	37	18	9	1	17	382
14.30 - 14.45	90	118	42	24	22	17	32	15	18	0	18	396
14.45 - 15.00	94	101	78	20	24	16	31	19	16	1	19	419
15.00 - 15.15	82	105	69	29	18	14	28	20	9	0	18	392
15.15 - 15.30	75	103	53	20	26	7	20	15	7	0	17	343
15.30 - 15.45	80	146	58	22	23	15	44	23	14	0	16	441
15.45 - 16.00	90	106	65	25	21	26	29	11	10	7	19	409
16.00 - 16.15	96	94	79	42	15	22	39	34	23	1	16	461
16.15 - 16.30	85	108	87	38	14	13	33	22	14	0	14	428
16.30 - 16.45	130	123	98	35	19	12	27	25	13	0	18	500
16.45 - 17.00	109	108	63	18	15	5	23	23	15	1	15	395

Table B- 2: Flow data of location: Loc_41 (Continued)

Time	Motorcycle	Three-wheeler	Car	Van	Utility Vehicle	Light Goods Vehicle	Medium Goods Vehicle	Heavy Goods Vehicle	Multi Axle Vehicle	Mini Bus	Large Bus	Total
17.00 - 17.15	96	117	102	16	14	6	20	15	16	2	17	421
17.30 - 17.45	85	118	66	22	25	9	28	27	16	1	19	416
17.45 - 18.00	88	105	92	22	19	11	17	9	10	2	30	405
18.00 - 18.15	76	84	66	30	15	7	29	13	11	2	26	359
18.15 - 18.30	116	90	57	32	21	6	23	10	8	0	13	376
18.30 - 18.45	73	83	66	28	14	8	28	15	9	3	17	344
18.45 - 19.00	64	124	75	31	24	14	19	16	10	2	16	395
19.00 - 19.15	93	122	51	24	12	13	15	13	8	2	13	366
19.15 - 19.30	108	109	56	38	17	19	24	22	13	1	8	415
19.30 - 19.45	71	108	56	26	24	5	9	12	11	2	6	330
19.45 - 20.00	46	101	99	26	15	15	14	9	23	0	9	357
20.00 - 20.15	88	131	70	32	14	5	16	9	6	0	5	376
20.15 - 20.30	76	128	64	27	13	6	20	18	21	1	12	386
20.30 - 20.45	80	116	53	21	11	9	11	14	15	0	10	340
20.45 - 21.00	42	96	59	18	12	7	9	24	13	1	4	285
21.00 - 21.15	33	87	55	23	13	9	11	20	10	1	3	265
21.15 - 21.30	26	94	61	25	10	8	10	13	4	0	1	252
21.30 - 21.45	20	93	38	22	12	5	11	14	8	2	2	227
21.45 - 22.00	22	84	55	23	13	9	14	8	5	1	4	238

APPENDIX C: Data Collection Location Data

A summary of the geometric details of the study locations are shown in Table C- 1.

Table C- 1: Summary of geometric details of study locations

Location code	Median type	Number of lanes per direction	Lane width (m)	Effective lane width (m)	Shoulder type	Shoulder width (m)	Lateral Clearance (m)	Built Environment	Access road density
Loc_1	Median Separated	2	3.5	3.5	Curb	-	2	Rural	0
Loc_2	Median Separated	2	3.2	3.2	Hard Shoulder	1.5	4	Rural	1
Loc_3	Median Separated	2	3.3	3.3	Hard Shoulder	3	3	Sub-Urban	7
Loc_4	Median Separated	2	3.3	3.3	Hard Shoulder	3	3	Sub-Urban	11
Loc_5	Median Separated	2	3.5	3.5	Curb	-	4	Rural	0
Loc_6	Median Separated	3	3	3	Curb	-	0.5	Rural	0
Loc_7	Median Separated	3	3	3	Curb	-	0.5	Rural	2
Loc_8	Divided	2	2.8	2.35	Curb	-	1.5	Sub-Urban	1
Loc_9	Divided	2	2.8	2.8	Curb	-	2	Rural	1
Loc_10	Median Separated	2	3.5	3.5	Curb	-	2	Sub-Urban	6
Loc_11	Divided	2	2.8	2.8	Curb	-	2.5	Rural	1
Loc_12	Divided	2	2.8	2.8	Curb	-	2.5	Rural	0
Loc_13	Median Separated	2	3.5	3.5	Curb	-	4	Rural	1
Loc_14	Median Separated	2	3.5	3.5	Curb	-	4	Rural	0

Table C- 1: Summary of geometric details of study locations (Continued)

Location code	Median type	Number of lanes per direction	Lane width (m)	Effective lane width (m)	Shoulder type	Shoulder width (m)	Lateral Clearance (m)	Built Environment	Access road density
Loc_15	Median Separated	4	3	2.55	Curb	-	1.5	Urban	4
Loc_16	Divided	2	3.4	3.4	Curb	-	2.5	Sub-Urban	2
Loc_17	Divided	2	2.8	2.8	Curb	-	2	Rural	1
Loc_18	Divided	2	3.4	2.5	Curb	-	2.5	Sub-Urban	4
Loc_19	Divided	2	3.4	2.5	Curb	-	2.5	Sub-Urban	4
Loc_20	Median Separated	3	3.2	3.2	Curb	-	2.5	Sub-Urban	5
Loc_21	Median Separated	3	3	3	Curb	-	2.5	Sub-Urban	6
Loc_22	Median Separated	2	3.2	3.2	Hard Shoulder	3	3.5	Urban	3
Loc_23	Median Separated	2	3.3	3.3	Hard Shoulder	2	2	Urban	1
Loc_24	Median Separated	2	3.3	3.3	Hard Shoulder	2	2	Urban	5
Loc_25	Median Separated	2	3.3	2.4	Curb	-	1	Urban	13
Loc_26	Median Separated	2	3	2.1	Curb	-	2	Urban	6
Loc_27	Median Separated	2	3.2	2.3	Curb	-	1	Urban	10
Loc_28	Median Separated	2	3.5	3.5	Curb	-	4	Rural	0
Loc_29	Median Separated	2	2.8	2.8	Hard Shoulder	1.5	2	Urban	4

Table C- 1: Summary of geometric details of study locations (Continued)

Location code	**Median type**	**Number of lanes per direction**	**Lane width (m)**	**Effective lane width (m)**	**Shoulder type**	**Shoulder width (m)**	**Lateral Clearance (m)**	**Built Environment**	**Access road density**
Loc_30	Median Separated	2	2.8	2.8	Hard Shoulder	1.5	2	Urban	6
Loc_31	Median Separated	2	2.8	2.8	Curb	-	3	Urban	5
Loc_32	Median Separated	2	3	2.55	Soft Shoulder	2	2	Sub-Urban	2
Loc_33	Median Separated	2	3	2.1	Hard Shoulder	1	1	Sub-Urban	10
Loc_34	Divided	2	3.4	2.5	Curb	-	2.5	Sub-Urban	2
Loc_35	Median Separated	2	4	4	Curb	-	2	Sub-Urban	1
Loc_36	Median Separated	2	4	4	Curb	-	2	Sub-Urban	1
Loc_37	Divided	2	3	2.55	Curb	-	1.5	Sub-Urban	0
Loc_38	Divided	2	3	2.55	Curb	-	1.5	Sub-Urban	1
Loc_39	Median Separated	2	3	3	Curb	-	2	Sub-Urban	10
Loc_40	Median Separated	2	3.5	3.5	Curb	-	2	Sub-Urban	6
Loc_41	Median Separated	2	3.4	3.4	Hard Shoulder	3	3.5	Sub-Urban	5
Loc_42	Median Separated	2	3.4	3.4	Hard Shoulder	3	3.5	Sub-Urban	5
Loc_43	Median Separated	2	3.4	3.4	Hard Shoulder	3	3.5	Sub-Urban	6
Loc_44	Median Separated	2	3.2	2.75	Curb	-	1.5	Sub-Urban	4

Table C- 1: Summary of geometric details of study locations (Continued)

Location code	Median type	Number of lanes per direction	Lane width (m)	Effective lane width (m)	Shoulder type	Shoulder width (m)	Lateral Clearance (m)	Built Environment	Access road density
Loc_45	Divided	2	3	2.1	Curb	-	2	Urban	3
Loc_46	Divided	2	2.8	2.35	Curb	-	1.5	Sub-Urban	1
Loc_47	Median Separated	2	3	2.1	Curb	-	2	Urban	4
Loc_48	Divided	2	3	2.1	Curb	-	2	Urban	3
Loc_49	Median Separated	2	3.4	3.4	Hard Shoulder	3	3.5	Sub-Urban	6
Loc_50	Median Separated	2	4	4	Curb	-	1	Sub-Urban	4
Loc_51	Median Separated	2	4	4	Curb	-	1	Sub-Urban	4
Loc_52	Median Separated	3	3.3	2.7	Curb	-	1.5	Sub-Urban	5
Loc_53	Median Separated	3	3.3	2.7	Curb	-	1.5	Sub-Urban	5
Loc_54	Median Separated	2	3	3	Curb	-	2.5	Sub-Urban	10
Loc_55	Divided	2	3	2.1	Curb	-	2	Urban	5
Loc_56	Divided	2	3	2.1	Curb	-	2	Urban	5
Loc_57	Divided	2	4	4	Curb	-	2	Urban	6
Loc_58	Median Separated	2	3.2	2.3	Curb	-	1	Urban	6
Loc_59	Median Separated	2	3.2	2.3	Curb	-	1	Urban	6
Loc_60	Divided	2	4	4	Curb	-	2	Sub-Urban	5
Loc_61	Median Separated	2	2.8	2.8	Curb	-	3	Urban	13

Table C- 1: Summary of geometric details of study locations (Continued)

Location code	Median type	Number of lanes per direction	Lane width (m)	Effective lane width (m)	Shoulder type	Shoulder width (m)	Lateral Clearance (m)	Built Environment	Access road density
Loc_62	Median Separated	2	3.3	2.4	Curb	-	1	Urban	11
Loc_63	Median Separated	2	3.3	2.4	Curb	-	1	Urban	11
Loc_64	Median Separated	2	3.2	2.3	Curb	-	1	Urban	10
Loc_65	Median Separated	2	3.3	3.3	Hard Shoulder	2	2	Urban	3
Loc_66	Median Separated	2	3.3	3.3	Hard Shoulder	2	2	Urban	3
Loc_67	Median Separated	2	3.3	2.4	Curb	-	1	Urban	13
Loc_68	Median Separated	2	3.2	3.2	Curb	-	2.5	Sub-Urban	8
Loc_69	Median Separated	2	3.2	3.2	Curb	-	2.5	Sub-Urban	7
Loc_70	Median Separated	3	3.2	3.2	Curb	-	2.5	Sub-Urban	6
Loc_71	Median Separated	3	3.2	3.2	Curb	-	2.5	Urban	4
Loc_72	Median Separated	3	3	3	Curb	-	2.5	Sub-Urban	6
Loc_73	Median Separated	3	3	3	Curb	-	2	Sub-Urban	5
Loc_74	Median Separated	2	2.8	2.8	Curb	-	3	Urban	5
Loc_75	Median Separated	2	3	3	Curb	-	2.5	Urban	5

Table C- 1: Summary of geometric details of study locations (Continued)

Location code	Median type	Number of lanes per direction	Lane width (m)	Effective lane width (m)	Shoulder type	Shoulder width (m)	Lateral Clearance (m)	Built Environment	Access road density
Loc_76	Median Separated	2	3	3	Curb	-	2.5	Urban	6
Loc_77	Median Separated	3	3.2	3.2	Curb	-	2.5	Urban	8
Loc_78	Median Separated	3	3	3	Curb	-	1.5	Sub-Urban	3
Loc_79	Median Separated	3	3	3	Curb	-	1.5	Sub-Urban	6
Loc_80	Median Separated	2	3.2	3.2	Curb	-	2	Sub-Urban	6
Loc_81	Median Separated	3	3	2.4	Curb	-	1	Urban	5
Loc_82	Median Separated	3	3	2.4	Curb	-	1	Urban	5
Loc_83	Median Separated	4	3	3	Curb	-	2.5	Sub-Urban	1
Loc_84	Median Separated	4	3	3	Curb	-	2.5	Sub-Urban	1
Loc_85	Median Separated	2	3.2	3.2	Curb	-	2	Sub-Urban	1

APPENDIX D: Free Flow Speed (FFS) model development

The FFS model development process is explained here. The roadway characteristics that influence FFS were selected using the ‘Backward regression’ method facilitated by the SPSS software which eliminates insignificant predictor variables from the model.

By doing so Lateral clearance, Median type and Built Environment were found to be statistically significant predictors of the FFS. Here the Lateral Clearance is added to the software as a ‘scale’ variable while the Median Type and Built Environment variables are added as nominal variables. The Built Environment categorical variable is dummy coded as explained in section 5.2.1. The variables eliminated include access point density, shoulder width and lane width.

Table D- 1 shows the regression model coefficient of determination (R^2) which is 0.59 indicates that a major portion of the variance of the dependent variable is described by the predictor variables. Further, it is observed that the independent variables

Table D- 1: FFS regression model summary

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.767[a]	.588	.551	6.90222

a. Predictors: (Constant), Sub_Urban, Median Type, Lateral_clearance, Urban

ANOVA[a]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	3053.692	4	763.423	16.025	.000[b]
	Residual	2143.828	45	47.641		
	Total	5197.520	49			

a. Dependent Variable: FFS

b. Predictors: (Constant), Sub_Urban, Median Type, Lateral_clearance, Urban statistically significantly predict Free Flow Speed as the P value is less than 0.05 as highlighted in the ANOVA table.

Table D- 2 portrays the coefficients of the predictor variables and their significance values which are all less than 0.1 indicating that the predictor variables can be accepted to the model at 90% confidence.

Table D- 2: FFS model regression coefficients

Coefficients[a]

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.
1	(Constant)	34.698	5.103		6.800	.000
	Median Type	5.982	3.389	.176	1.765	.084
	Lateral_clearance	4.541	1.335	.376	3.401	.001
	Urban	-15.393	3.522	-.692	-4.371	.000
	Sub_Urban	-13.389	3.182	-.654	-4.208	.000

a. Dependent Variable: FFS

Hence the FFS model can be written as shown in equation (D.1)

$$FFS = 35 + 6\, S_M + 4.5\, S_{LC} - 15.5\, S_{BE} \qquad \text{(D.1)}$$

Where,

FFS = Free Flow Speed (km/h)

S_M = Median type

S_{LC} = Lateral clearance (m)

S_{BE} = Built environment (Rural, Sub-Urban and Urban)

Since the built environment is entered as a categorical variable the model is modified in such a way that the variable (S_{BE}) explains all three categories rural, sub-urban and urban. Hence the values 0, 0.9 (=13.4/15.4), and 1 will be used to indicate rural, sub-urban and urban sections respectively. Similarly, for the 'Median Type (S_M)' variable, values 0 or 1 should be substituted for Divided (median-less) and Median separated sections respectively.

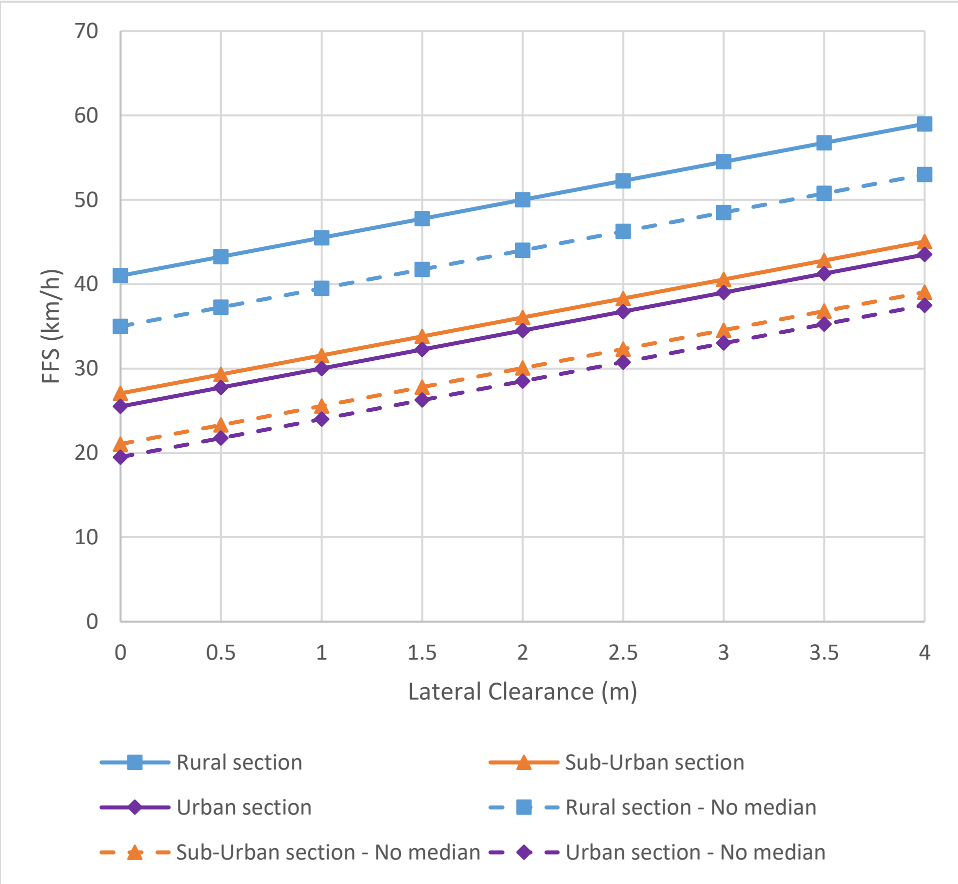

Figure D- 1: FFS Curves from model (CM – Center median)

Figure D- 1 illustrates the variation in FFS with Lateral clearance. The solid lines show the variation when the road is median separated whereas the dashed lines show the FFS variation when there is no median separation. It is observed from the model that typically the FFS reduces by a magnitude of 6km/h when there is no median separation between opposing traffic flows. The vehicles move at slower speeds because of the (potential) threat of opposing traffic moving to their lane for overtaking purposes or turning movements. Another observation is the drop of FFS with the roadside built environment changes from Rural > Sub-Urban > Urban. The drop from rural to sub-urban is significant, it being a drop of approximately 13.5km/h. A further drop of approximately 2km/h is FFS is observed from Sub-Urban to Urban sections. This may be due to the friction encountered by vehicles due to the roadside activities that arise due to developments along the road.

Lateral clearance is another factor that influenced the FFS of vehicles. As observed in Figure D- 1 with the increase in Lateral clearance the FFS increases by a factor of 4.5km/h per meter. Interestingly the lane width was not found to significantly influence the FFS of vehicles during the analysis.

Table D- 3 presents a summary of the model developed FFS values against the factors that affect it.

Table D- 3: Summary of model developed FFS values

Lateral Clearance (m)	FFS (km/h)					
	No median Separation			Median Separated		
	Rural	Sub-Urban	Urban	Rural	Sub-Urban	Urban
0.0	35	21	20	41	27	26
0.5	37	23	22	43	29	28
1.0	40	26	24	46	32	30
1.5	42	28	26	48	34	32
2.0	44	30	29	50	36	35
2.5	46	32	31	52	38	37
3.0	49	35	33	55	41	39
3.5	51	37	35	57	43	41
4.0	53	39	38	59	45	44